



Department of  
Transportation

# **HIGHWAY DESIGN MANUAL**

## **Chapter 5**

### **Basic Design**

**Revision 101  
(Limited Revision)**

**May 15, 2023**

**Issued by Engineering Bulletin 23-014  
Effective with Design Approval on or after Sept 1, 2023**

Section	Changes
5.2.1.2.E	Clarifies where to place traffic data collected for projects for use by Highway Data Services Bureau

## TABLE OF CONTENTS

<b>5.1 INTRODUCTION.....</b>	<b>6</b>
5.1.1 Project Development & Public Involvement.....	6
5.1.2 Nonconforming Features.....	6
<b>5.2 SPEED STUDIES, HIGHWAY CAPACITY, AND LEVEL OF SERVICE.....</b>	<b>8</b>
5.2.1 Traffic Data.....	8
5.2.2 Traffic Flow Diagrams, Growth Rates and Diversion Analysis.....	11
5.2.3 Capacity Analysis.....	15
5.2.4 Updating Traffic Data and Capacity Analysis.....	19
5.2.5 Speed Studies.....	19
<b>5.3 CRASH ANALYSIS.....</b>	<b>23</b>
5.3.1 Applicability.....	23
5.3.2 Timing and Responsibility.....	23
5.3.3 Crash Analysis Procedures.....	24
5.3.4 Highway Safety Investigation Report.....	24
<b>5.4 VACANT.....</b>	<b>25</b>
<b>5.5 RIGHT OF WAY (ROW).....</b>	<b>25</b>
5.5.1 Abstract Request Maps.....	25
5.5.2 Design Approval Document (DAD) and Preliminary Plans.....	25
5.5.3 Right of Way Determination.....	26
5.5.4 Taking Line Review Meeting.....	27
5.5.5 Design Phases V-VI, Final Design Stage.....	28
5.5.6 Types of Right of Way and Access.....	30
5.5.7 Encroachments.....	36
5.5.8 Excess Right of Way.....	36
5.5.9 ROW Markers.....	37
<b>5.6 ECONOMIC ANALYSIS.....</b>	<b>39</b>
<b>5.7 DESIGN ELEMENTS.....</b>	<b>44</b>
5.7.1 Design Vehicle.....	44
5.7.2 Sight Distance.....	46
5.7.3 Horizontal Curves.....	47
5.7.4 Vertical Alignment.....	64
5.7.5 Climbing Lanes.....	67
5.7.6 Emergency Escape Ramps.....	67
5.7.7 Travel Lanes and Shoulders.....	68
5.7.8 Lane Drops.....	68
5.7.9 Medians.....	69
5.7.10 Median - Emergency Crossovers.....	69
5.7.11 Roadway Clear Zone.....	70
5.7.12 Vertical and Horizontal Clearances.....	70
5.7.13 Rollover.....	74
5.7.14 Bridge Roadway Width.....	75
5.7.15 Passive Snow Control.....	75
5.7.16 Parking.....	77
5.7.17 Access Control.....	80
5.7.18 Driveways.....	83
5.7.19 Frontage Roads - Service Roads.....	83
5.7.20 High-Occupancy Vehicle (HOV) Lanes.....	83
5.7.21 Complete Streets.....	83

<b>5.8 DESIGN CONSIDERATIONS.....</b>	<b>86</b>
5.8.1 Driver Expectancy .....	86
5.8.2 Geometric Considerations .....	86
5.8.3 Joint Use of the Highway Corridor .....	87
5.8.4 Social, Economic, and Environmental Considerations.....	87
5.8.5 Utilities.....	88
5.8.6 Increasing Capacity Without Adding Lanes.....	88
5.8.7 Traffic Calming.....	90
5.8.8 Aesthetics .....	91
<b>5.9 INTERSECTIONS AT GRADE.....</b>	<b>92</b>
5.9.1 Types of Intersections.....	92
5.9.2 Intersection Capacity and Level-of-Service Analysis .....	100
5.9.3 Intersection Geometrics .....	103
5.9.4 Principles of Channelization .....	109
5.9.5 Intersection Sight Distance.....	116
5.9.6 Access Control on Uncontrolled Access Facilities.....	120
5.9.7 Signalization .....	120
5.9.8 Intersection Widening.....	120
5.9.9 Bus Stops/Turnouts.....	132
5.9.10 Divided Highway Median Openings in Urban Areas .....	134
<b>5.10 REFERENCES.....</b>	<b>138</b>

<b>APPENDIX A - NYSDOT POLICY AND STANDARDS FOR THE DESIGN OF ENTRANCES TO STATE HIGHWAYS.....</b>	<b>5A</b>
<b>APPENDIX B - VERTICAL HIGHWAY ALIGNMENT SIGHT DISTANCE CHARTS.....</b>	<b>5B</b>
<b>APPENDIX C - INTERSECTION SIGHT DISTANCE CHARTS.....</b>	<b>5C</b>
<b>APPENDIX D - LEVEL OF SERVICE AND CAPACITY ANALYSIS ON STATE HIGHWAYS.....</b>	<b>5D</b>
<b>APPENDIX E - DESIGN OF TOLLING FACILITIES .....</b>	<b>5E</b>

## LIST OF EXHIBITS

EXHIBIT 5-1A	EXAMPLE DESIGN HOURLY VOLUME AS FUNCTION OF AADT .....	9
EXHIBIT 5-1B	DESIGN YEAR BASED ON PROJECT TYPE <sup>3,4</sup> .....	13
EXHIBIT 5-2	SPEED STUDY LOCATIONS .....	22
EXHIBIT 5-5	ECONOMIC ANALYSIS PROBLEM TYPES, ANALYSIS METHODS, GUIDANCE IN FORCE, AND CONTACT FOR MORE INFORMATION .....	39
EXHIBIT 5-6	HORIZONTAL SIGHT DISTANCE .....	47
EXHIBIT 5-7	SIDE FRICTION FACTOR .....	49
EXHIBIT 5-8	RECOMMENDED SPEED ON HORIZONTAL CURVES TO AVOID SKIDDING (LOW SPEED) .....	51
EXHIBIT 5-9	RECOMMENDED SPEED ON HORIZONTAL CURVES TO AVOID SKIDDING .....	52
EXHIBIT 5-10	RECOMMENDED SPEED ON HORIZONTAL CURVES TO AVOID TRUCK ROLLOVERS (LOW SPEED) .....	54
EXHIBIT 5-11	RECOMMENDED SPEED ON HORIZONTAL CURVES TO AVOID TRUCK ROLLOVERS .....	54
EXHIBIT 5-12	MAXIMUM RELATIVE GRADIENT .....	56
EXHIBIT 5-13	METHOD OF ATTAINING SUPERELEVATION .....	57
EXHIBIT 5-14	METHOD OF ATTAINING SUPERELEVATION – FOUR LANE DIVIDED HIGHWAY* .....	58
EXHIBIT 5-15	SUPERELEVATION RUNOFF $L_R$ (FT) FOR HORIZONTAL CURVES .....	60
EXHIBIT 5-16	CROSS SECTION VIEW OF TRAVEL LANE WIDENING ALONG SPIRAL CURVES .....	62
EXHIBIT 5-17	PLAN VIEW OF TRAVEL LANE WIDENING ALONG CURVES WITHOUT SPIRAL TRANSITIONS .....	63
EXHIBIT 5-18	MINIMUM MEDIAN WIDTHS .....	69
EXHIBIT 5-19	HORIZONTAL CLEARANCE ROAD SECTIONS .....	72
EXHIBIT 5-20	MINIMUM HORIZONTAL CLEARANCES .....	73
EXHIBIT 5-21	MAXIMUM ROLLOVER RATES .....	75
EXHIBIT 5-22	HIGHWAY ACCESS ISSUES .....	82
EXHIBIT 5-23	ROUNDBOUT INTERSECTIONS .....	97
EXHIBIT 5-24	DIVIDED HIGHWAY CROSSINGS AND OFFSET INTERSECTIONS	
EXHIBIT 5-25	JUG HANDLES AND INDIRECT LEFT TURNS .....	97
EXHIBIT 5-26	CONTROL DELAY AND LEVEL OF SERVICE (LOS) .....	101
EXHIBIT 5-27	CROSS SLOPES FOR INTERSECTING HIGHWAYS .....	107
EXHIBIT 5-27A	CURB AND BARRIER TREATMENTS FOR PEDESTRIAN REFUGE ISLANDS <sup>1</sup> .....	111
EXHIBIT 5-28	HIGH-CAPACITY SIGNALIZED INTERSECTION WITH DOUBLE LEFT-TURN LANES AND RIGHT- TURNING ROADWAYS .....	114
EXHIBIT 5-29	APPROACH AND DEPARTURE SIGHT TRIANGLES .....	117
EXHIBIT 5-30	INTERSECTION SIGHT DISTANCE QUICK REFERENCE .....	119
EXHIBIT 5-31	INTERSECTION WIDENING FOR HEAVY THROUGH TRAFFIC .....	123
EXHIBIT 5-32	SHADOWING LEFT-TURN LANES .....	128
EXHIBIT 5-33	DECELERATION DISTANCES (FT) FOR PASSENGER CARS APPROACHING INTERSECTIONS (BRAKING AT A COMFORTABLE RATE OF $5.8 \text{ ft/s}^2$ ( $1.8 \text{ m/s}^2$ )) .....	130
EXHIBIT 5-34	SAFETY WIDENING AT T-INTERSECTION ON RURAL TWO-LANE ROAD .....	133
EXHIBIT 5-35	LEFT-TURN SLOT WITH DIVIDER ON RIGHT .....	136
EXHIBIT 5-36	DESIGN OF MEDIAN LANES FOR MEDIANS OVER 25 FT (8 M) WIDE .....	137

## 5.1 INTRODUCTION

The Department is committed to developing projects that improve the movement of people and goods, while recognizing community needs and values. Projects should be safe, serviceable, constructible, economical to build and maintain, and in harmony with the community and its environmental, scenic, cultural, and natural resources. Successful designs result from a careful balance of safety, mobility, and capacity needs with social, economic, and environmental needs.

This chapter provides guidance regarding the basic elements of highway design to designers and other project developers. The information presented is not all-inclusive, but must be used in conjunction with information found in other chapters of this manual and documents adopted by the Department to achieve the most appropriate design meeting the goals and objectives of the project.

### 5.1.1 Project Development & Public Involvement

There are various phases of development through which a project design must evolve. These phases are described in the [Project Development Manual](#). Projects should be progressed through these phases with the aid and advice of project stakeholders, which include the Regional Functional Units, the Main Office, the public, and appropriate advisory and regulatory agencies. Early, effective, and continuous stakeholder involvement fosters meaningful participation and sense of ownership in the project development process. The open exchange of information and concerns between the Department and stakeholders benefits projects by identifying key issues early in the process, developing consensus for project solutions, and building trust among stakeholders.

### 5.1.2 Nonconforming Features

During the project development process, design element trade-offs are routinely considered. Quantitative measures are to be used, whenever feasible, to compare and evaluate the effects of trade-offs. When the Department evaluates such trade-offs in the course of considering transportation needs and community needs, public safety (whether driving, riding, walking, or bicycling) remains the foremost issue to consider.

Variances from standard values established for the critical design elements listed in [Chapter 2](#) of this manual require a justification and approval as described in that chapter. In addition to the critical design elements, there are other design elements with established values or parameters that must be considered when scoping and designing a project. These other elements are important because they can have a considerable effect on the cost, scope, schedule, and quality of a project. Any decisions to vary from recommended values or accepted practices for these elements need to be explained and documented as nonconforming features in the scoping and design approval documents and, when identified after design approval, in the project files. The more significant the deviation or the more important an element is to quality design, the more detailed the explanation will be. For example, an explanation similar in detail to the requirements for nonstandard features is appropriate if the Department proposes to build an acceleration lane to 75% of the values in AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, or not attain the compound curve ratio. However, not achieving the minimum length of a superelevation runoff by a few feet would only warrant a brief explanation in the report.

The following is a listing of some of the other elements which are described in detail in this and other chapters. It is being provided to give a representative sample of items to be considered when scoping and designing a project. It is not in priority order or intended to be all-inclusive.

- Level of service
- Median width
- Minimum pipe size
- Sag vertical curve (Sag vertical curve sight distance is a critical design element where sight lines are restricted under bridges or other vertical sight obstructions. Refer to Section 5.7.4.2.B)
- Minimum length of vertical curves
- Lane drops
- Tapers for lane drops
- Driveway grade
- Driveway opening
- Driveway spacing
- Buffer zone for snow storage
- Width of spread for ponding water
- Clear zone
- Objects within the clear zone
- Intersection radii (including accommodation of identified oversized vehicles)
- Intersection and decision sight distance
- Superelevation runoff/runout length
- Broken back curves
- Compound curves
- Auxiliary lane lengths
- Adequate provisions for pedestrians and bicyclists (refer to Chapters 17 and 18 of this manual)
- Transit and high-occupancy vehicle facilities and accommodations
- Design storm for drainage facilities (refer to Chapter 8 of this manual)
- Curbing
- Guide rail
- Median barrier
- Longitudinal rumble strips
- Horizontal clearance
- Permanent and temporary soil erosion and sediment control

A checklist of common nonconforming features is provided on the Chapter 5 Internet page at: <https://www.dot.ny.gov/divisions/engineering/design/dgab/hdm/chapter-5>.

## 5.2 SPEED STUDIES, HIGHWAY CAPACITY, AND LEVEL OF SERVICE

Traffic data and a capacity analysis are used to develop the geometric design, evaluate alternatives, design traffic signals, etc. The data collection and analysis depends on the project type, highway functional class, and the presence of cross roads or major driveways. Refer to Section 5.2.3 of this chapter for guidance on using older data and capacity analyses.

### 5.2.1 Traffic Data

#### 5.2.1.1 Study Area

As a minimum, the study area for the traffic analysis should extend one interchange or major intersection before and after the limits of the proposed work to capture detours and diversions during and after construction. These include:

- Highway.
- All approaches of intersections and driveways/entrances with one-way volumes of 100 vehicles per hour (vph) or more.
- Ramps.
- Weaving sections.
- Merges and diverges.
- Service roads and frontage roads.

For projects where substantial diversion or more extensive detours may be needed, the study area should be expanded to enable an analysis of the effects. In general, highways that will have an additional 30 vehicles per hour per lane or more should be included. A combination of microscopic and mesoscale analysis may be used for large study areas.

#### 5.2.1.2 Data Acquisition

##### A. Data for Simplified Capacity Analysis

Secondary traffic data (i.e., data that was not obtained specifically for the project) may be used for:

- Projects on routes with little delay (LOS B or better). The level of service (LOS) should be observed during peak periods, which may include:
  - The weekday AM and PM peaks.
  - Saturday noon-hour peaks near shopping areas or malls.
  - Friday and Sunday nights on summer recreation routes.
  - Saturday and Sunday AM and PM peaks near ski areas.
  - Immediately before and after regular sporting events, concerts, or other special events.
- Maintenance-type projects (e.g., pavement preventive maintenance and bridge preventive maintenance projects).
- Construction lane closures or detours. Secondary traffic data includes the annual average daily traffic (AADT) data from the NYSDOT [Highway Traffic Data Viewer](#). Additional sources may be available from the Highway Data Services Bureau and



the Regional Planning Group. *The Highway Capacity Manual*, Regional data, or Exhibit 5-1a (below), and the *Traffic Engineering Handbook*, can be used to obtain the design hourly volume (DHV), directional design hourly volume (DDHV), and any other required traffic data.

#### Exhibit 5-1a Example Design Hourly Volume as Function of AADT

AADT	Average DHV as a % of AADT
0 – 2,500	15.1%
2,500 – 5,000	13.6%
5,000 – 10,000	11.8%
10,000 – 20,000	11.6%
20,000 – 50,000	10.7%
50,000 – 100,000	9.1%
100,000 – 200,000	8.2%
> 200,000	6.7%

*Highway Capacity Manual, 6<sup>th</sup> Edition, TRB, p. 3-12, Exhibit 3-11.*

#### B. Pedestrian and Bicycle Traffic

For most projects, the “Capital Projects Complete Streets Checklist”, as described in [Chapter 18](#), is used to identify existing, latent, seasonal, and planned pedestrian traffic needs. However, pedestrian traffic data acquisition may be necessary to determine the appropriate treatments and design of pedestrian facilities in areas of high pedestrian volumes and/or special use areas, e.g., central business and walking districts, colleges, amusement parks, etc. Pedestrian data acquisition can be accomplished through pedestrian counts, pedestrian questionnaires, and pedestrian origin and destination studies. For information on pedestrian data acquisition, refer to the following:

- *Sketch-Plan Method for Estimating Pedestrian Traffic for Central Business Districts and Suburban Growth Corridors:*  
<http://trrjournalonline.trb.org/doi/abs/10.3141/1578-06>
- *Guidebook on Methods to Estimate Non-Motorized Travel, Volumes 1 & 2:*  
[http://safety.fhwa.dot.gov/ped\\_bike/docs/guidebook1.pdf](http://safety.fhwa.dot.gov/ped_bike/docs/guidebook1.pdf)  
[http://safety.fhwa.dot.gov/ped\\_bike/docs/guidebook2.pdf](http://safety.fhwa.dot.gov/ped_bike/docs/guidebook2.pdf)

#### C. Data Requirements for Projects with Potential Capacity Measures

Where crowd sourced data from cell phones, GPS units, etc. will be used, traffic counts are required along select segments of the highway network to calibrate the data. A standardized format for turning movement data is provided on the Highway Design Manual Chapter 5 webpage. Additionally, the following traffic data will generally be required to perform a full capacity analysis. The Transportation Research Board's *Highway Capacity Manual* (HCM) or software program should be referenced to determine the specific traffic data and physical data required.

- Percentage of trucks, buses, and RVs
- Highway AADT
- Highway DHV (two-way)
- Highway Directional Design Hourly Volume (DDHV) (one-way)
- Highway two-way (design hour) percent trucks
- Ramp/turning roadway DHV
- Peak-hour factor
- Free-flow speed on highway and ramps
- Weaving volumes
- Ramp volumes and adjacent ramp volumes
- Parking and bus stops per hour
- Average travel speeds
- Bicycle DHV (order of magnitude to determine appropriate facility)
- Pedestrian DHV (for existing or proposed sidewalks, signalized intersections, and transit-related pedestrian facilities)

Traffic volumes are to be generated from:

- Full 24-hour 7-day count with class and speed
- Turn counts for at least two hours that encompass the peak hour\*
- Turn counts on Tuesday, Wednesday, or Thursday
- Turn counts while school is in session and no major events
- Turn counts on two separate days, if feasible
- Midday typically only in areas with commercial shopping

\* Some highways have a midday peak period that should be shown and considered in the project's geometric and traffic signal design. Also, there may be a need to give traffic volumes for other peak periods for commercial generators or special events (e.g., Saturday peak shopping hours, concert performances, fairs).

Calibration factors for the models are to be taken within the same time frame as the traffic counts. These include:

- Queue lengths during counts for calibration
- Travel time and delay runs using the following car method
- Signal timings
- Travel speeds

**Note:** A major intersection, for traffic count purposes, is a signalized intersection, an intersection approaching any of the warrants for signalization in accordance with the *Official Compilation of Codes, Rules and Regulations of the State of New York Title 17 Transportation (B) Chapter V (a.k.a., New York State Supplement to the Manual on Uniform Traffic Control Devices [NYS Supplement])*, or an intersection approaching the warrants for a turn lane as contained in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

For methods of gathering motorized traffic counts, refer to the Regional Planning Group, the *Traffic Engineering Handbook*, and [New York State Traffic Monitoring Standards for Short Count Data Collection](#).

#### E. Submissions of Collected Traffic Data to Highway Data Services Bureau

Upon collection, all traffic data collected is to be formatted in accordance with the templates on the [HDM Ch 5 web page](#) and posted in the Projectwise folder named "Traffic Data".

The following naming convention is to be used for files:

County\_Mile Marker\_Direction or Cross Road\_Collection Date (YYYYMMDD)

### 5.2.2 Traffic Flow Diagrams, Growth Rates and Diversion Analysis

#### 5.2.2.1 Diversion Analysis

A diversion analysis may be required when a significant change in a traffic network is proposed and alternate routes exist and are expected to be used.

Significant changes may include:

- Design changes to a facility that have a significant effect on capacity or LOS
- Addition of a facility
- Removal of a facility
- Development
- Long Term Construction (For guidance regarding construction-related diversions, see [HDM Chapter 16](#))

Diversions may result in an increase or a decrease in volumes on a facility. For example, diversions resulting in an increase in volume could occur when a facility increases in capacity and attracts volume from other facilities. Conversely, diversions resulting in a decrease in volume could occur when a facility's capacity is reduced and volume diverts to other route choices.

During project scoping, it should be determined whether the proposed project has the potential for traffic diversions. In these cases, the study area shall be determined based on alternate route choices and roadways/intersections that may be affected by the traffic diversions. The Regional Travel Demand Model, maintained by the MPO, or a dynamic traffic model using crowd sourced data such as cell phone or fleet GPS data should be used to generate revised roadway volumes and intersection turn counts to be used in the analyses of the project alternatives.

#### 5.2.2.2 Origin Destination Studies

An Origin–Destination (O-D) study may also be required when a significant change in a traffic network is proposed, regardless of how long the duration will be for this change. An O -D study may be desired in the absence of a Regional Travel Demand Model or to provide updated site-specific data to be utilized to update to the Regional Travel Demand Model.

O-D studies can be performed in a variety of ways, with surveys and vehicle tracking being the most common. A thorough explanation of many of these techniques is provided in a research study performed in Indiana: <https://trid.trb.org/view/864635>.

This research reviews the various techniques used to perform O-D studies. Topics of discussion include accuracy of data, general costs for some O-D studies previously performed, and selection of the O-D technique based on the study objectives.

It is also important to understand and document what elements require additional work and where it is not possible to perform this work given the unavailability of information or exorbitant cost of obtaining the information and performing the analysis. These considerations should be weighed during the early stages of the analysis process and should be discussed during scoping.

#### 5.2.2.3 Design Year Projections for Future Traffic

The projected traffic volumes for future years are to be determined using the existing traffic data, background growth rates and should include the added traffic volumes from planned development and other reasonably anticipated/foreseeable projects. Reasonably anticipated/foreseeable projects include:

- Projects that have had an environmental determination (e.g., SEQRA or NEPA Record of Decision), which includes all Department projects with Design Approval.
- Private development that has completed the SEQRA process or has started the SEQRA process and is very likely to complete the SEQRA process before the project letting.
- Public projects on the approved TIP and under design.

Contact the Regional Planning Group, MPO, and municipal planners, as needed for growth rates and the traffic volumes from planned development and reasonably anticipated/foreseeable projects.

The year selected as the Design Year for future traffic will assist in determining how well the project meets the objectives for capacity, delay and Level of Service over the expected service life of the project. Design years are based on the estimated time of completion (ETC), which is when the construction work is complete. The ETC is normally rounded up in 5 year increments to avoid revisions required by §5.2.4. Refer to the Exhibit 5-1b for the appropriate Design Year based on the project type. Refer to Chapter 2 of this manual, §2.2 for definitions of project type.

Design years for traffic forecasts other than shown in Exhibit 5-1b below shall not be used since they could result in an incomplete analysis that does not identify opportunities for operational improvements. To project future pedestrian or bicycle traffic volumes, refer to the documents referenced in Section 5.2.1.2.B.

**Exhibit 5-1b Design Year Based on Project Type<sup>3,4</sup>**

PROJECT WORK TYPE <sup>3,4</sup>		DESIGN YEAR
HIGHWAY WORK	Pavement Preventive and Corrective Maintenance (including 1R and 2R projects) <sup>1</sup>  Culvert Replacements and Culvert Rehabilitations (where the project scope is within that of a 1R or 2R project) <sup>1</sup>	Not Usually Required <sup>2</sup>
	3R - Resurfacing, Restoration, and Rehabilitation Minor Intersection Reconstruction	ETC+10
	Reconstruction and New Construction Major Intersection Reconstruction	ETC+20
	Tunnel	ETC+30
BRIDGE WORK	Element Specific Cyclical Bridge Work <sup>1,4</sup>	Not Usually Required <sup>2</sup>
	Element Specific Bridge Work <sup>1,4</sup>	Not Usually Required <sup>2</sup>
	Bridge Rehabilitation <sup>4,6</sup>	ETC+10 or ETC+ 20 <sup>5</sup>
	New and Replacement Bridges <sup>4,6</sup>	ETC+20
OTHER WORK & MISCELLANEOUS/SPECIAL WORK <sup>1</sup>		Not Usually Required <sup>2</sup>

Notes:

1. In some cases, these types of projects can be accomplished with minimal impacts to the traffic using the highway. However, closure of highway travel lanes generally requires an analysis to assure that the traffic lanes that remain open are sufficient to use for Work Zone Traffic Control during closure. ETC traffic should be used for the analysis and also to schedule work operations outside peak travel periods where feasible or determine the appropriate detour routes around the proposed highway work zone.
2. Design Year is not usually required for these types of projects as traffic analyses are not generally required. However, if the project proposes permanent changes that affect roadway capacity, then the designer should check at least ETC+10 traffic for any effects to capacity, delay or Level of Service
3. See Highway Design Manual Chapter 2, Section 2.2 for project work type definitions.
4. Refer to the NYS Bridge Manual for descriptions of Bridge Project Work Types.
5. For projects that include superstructure replacement, a design year traffic forecast of ETC+20 shall be used. For all other bridge rehabilitation projects, use a design year traffic forecast of ETC+10.
6. Where the project includes a bridge replacement or superstructure replacement in close proximity to an intersection, the future capacity needs of the intersection (through ETC+20) shall be analyzed and the bridge width shall accommodate the future width of the intersection as identified by the analysis.

#### 5.2.2.4 Proposed Signal Installations

The Estimated Time of Completion (ETC)+5 peak-hour turning movement volumes should be determined for proposed signal installations that will meet the signalization warrants in the design year, but do not meet the warrants for the ETC+0 year. The analysis of the ETC+5 traffic data can be used to determine if a signal should be installed as part of the project or in a future signal requirements contract. Regardless, the highway geometry (e.g., pavement width) should be designed to accommodate the proposed signal.

#### 5.2.2.5 Traffic Flow Diagrams

Traffic flow diagrams should be developed for the study peak hours (e.g., A.M., midday, P.M.) for existing, ETC, and the design year (typically ETC + 20). The diagrams should show:

- For each link, the current AADT, DHV, DDHV, and design-hour percent trucks.
- For all major intersections with crossroads or commercial driveways, the current design-hour turning movement volumes, design-hour percent trucks, and AADTs on all approaches for intersections.

Screen captures from traffic simulation software may be used, if legible.

For controlled and partially controlled segments, the traffic volumes in the diagram should be balanced to avoid vehicles disappearing and appearing mid-node during traffic simulations.

For uncontrolled access facilities, sections that are unbalanced by more than 10% should include side roads, major driveways, or a representative driveway to account for the vehicles entering and exiting the network mid-block.

## 5.2.3 Capacity Analysis

### 5.2.3.1 Capacity Analysis Requirements

Capacity analysis is a set of procedures used for estimating the traffic-carrying ability of facilities over a range of defined operational conditions. It provides tools to assess facilities and to plan and design improved facilities. Capacity analysis is performed using existing and projected (design year) design-hour traffic volumes for each alternative, including the no-build alternative.

For projects with an objective to reduce congestion, estimates of the existing and design-year vehicle hours of delay should be determined for the build and no-build alternatives. The results of the analysis should be included in the project's design approval document for evaluation of the various project alternatives.

For projects using a simplified capacity analysis per Section 5.2.1.2 A, the simplified analysis can be performed using the HCS or the Appendix D charts available on the [HDM Chapter 5 Internet page](#). Section 5.2.3.3 does not apply to simplified capacity analyses.

### 5.2.3.2 Capacity Analysis Methodology

Capacity analyses are to be consistent with the most recent version of the HCM. General announcements of the availability of HCM revisions will be made via Engineering Bulletins.

Department policy requires the designer to use capacity analysis software consistent with the HCM (however, see 5.2.3.7 below). For economic, efficiency, and quality assurance purposes, the Department preapproves a limited number of software programs for general use. The approved software programs and contact persons are shown on the "Department Approved List of Traffic Analysis Software Programs" on the webpage for [Chapter 5](#). Before running the software, designers should apply the latest patches or updates linked on the Department's Internet site to help ensure the software produces reasonably accurate results.

The same software should be used for all alternatives when practical. A roundabout, complex interchange or freeway alternative may necessitate separate software programs to be used.

When microsimulation analysis is performed, the setup and calibration of the model should follow the guidelines outlined in the [Traffic Analysis Toolbox, Volume III: Guidelines for Applying Traffic Microsimulation Software](#) (FHWA, 2004) and the HCM. Some target parameters for a microsimulation analysis include

- At least 10 runs with different random seeds.
- Run seeding for 15 min. (30 min. if the LOS is D or less).
- Run for 1 hour or until the queue lengths diminish (whichever is worse). In urban areas, a minimum run of 2 hours should be used.
- Calibrate to match existing observed conditions.
- Traffic signals should be optimized for no-build alternatives, and models should include traffic from approved development.

### 5.2.3.3 Calibration

Calibration is needed to verify that that model can reasonably predict the existing conditions and can be relied on to accurately portray future conditions. Calibration factors for the models (which are to be taken within the same time frame as the traffic counts) include:

- Queue lengths
- Travel speeds
- Delays

Latent demand occurs when vehicles are not able to enter the model. Generally, a high latent demand indicates that the model extents need to be extended to capture the demand. Latent delay occurs as a result of the latent demand; vehicles that are not able to enter the model experience delay outside of the model extents.

A calibration report or section is to be provided in the Traffic Impact Study. A sample calibration report is available at:

<https://www.dot.ny.gov/divisions/engineering/design/dgab/hdm/chapter-5>.

The queue lengths should be calibrated to within 20% for queues over 1,500 ft. and to within 300 ft. (12 vehicles) for shorter queues. Travel speeds should be calibrated to within 10 mph. Delay runs should be calibrated so that 85% of the runs are within 1 minute. Discrepancies that are not resolved by adjusting the model require an explanation.

#### 5.2.3.4 Measures of Effectiveness

##### A. Level of Service (LOS)

Level of service is a qualitative measure describing operational conditions within a traffic stream, based on service measures such as speed and travel time, freedom to maneuver, traffic interruptions, comfort, and convenience. Levels of service are given letter designations, from A to F, with LOS A representing the best operating condition and LOS F the worst. Level of service is specifically described for various types of highways or portions of highways in the *Highway Capacity Manual*.

Character	Minimum for the Design Year
Rural	C
Urban	D

Some projects, especially in urban areas, may provide levels of service below those shown due to social, economic, and environmental and/or policy/intergovernmental decisions made during project scoping and design. Decisions for lesser levels of service are to be made as nonconforming features in accordance with Section 5.1 of this manual and explained as appropriate.

The correlation between volumes and level of service is not direct. Level of service computations based on volume may not accurately represent the traffic conditions on congested highway segments when vehicles are moving very slowly. The volume at LOS F can be the same as the volume for a higher level of service due to slow speeds and low throughput. Therefore, travel speeds are essential for assessing whether the traffic volumes reflect a forced-flow condition or a free-flow condition in potentially congested areas.



## B. Delay

Delay is a quantitative measure describing the additional time it takes to travel through a segment.

- Control delay is the additional time required to travel a segment due to stopping for a stop sign, traffic signal, etc., usually measured in seconds per vehicle. Control delay can be calculated using common analysis tools and is the basis for the LOS. Delay runs account for control delay.
- Geometric delay is the time required to negotiate added roadway curvature, additional travel distances, etc. regardless of other traffic and is usually measured in seconds per vehicle. Roundabouts create several seconds of geometric delay. Geometric delay requires calculations of travel times based on the geometry and anticipated off peak speeds. Delay runs account for geometric delay.
- Delay runs measure the total time for the average vehicle to travel a segment in each direction and is usually measured in minutes per vehicle. Simulation of delay runs require more complicated analysis tools and should be done on projects with LOS of D or less. Existing delay run simulations should be calibrated, typically using a following car technique.
- Total delay is typically the total additional hours for all of the vehicles in one day travelling through the segment. Total hours of delay is useful when comparing build and no build alternatives.

## C. Queue Lengths

A traffic queue is a line of stopped or very slowly travelling motorists waiting to proceed. Queue lengths are a quantitative measure of the traffic demand. Where alternate routes are available, the queue length is only a part of the total demand. Queue lengths should represent 95% to 100% of the maximum queue during the peak period.

- In saturated conditions, queue lengths are essential measures of unmet demand. A building queue indicates a worsening of the congestion and more demand than capacity. Queues can build up to create gridlock and delay at upstream intersections and interchanges that would otherwise be free flowing.
- In unsaturated conditions, queues that develop can be processed by an intersection within a single cycle.
- In free flow conditions, queue lengths are not measurable.

## D. Peak Period Travel Speeds

Travel speed is a quantitative measure of the mobility during peak periods and can help determine route choices and diversion rates. This measure is useful when the LOS is D or less and is essential for origin and destination studies.

Where the existing mainline level of service is D or worse (refer to Section 5.2.3.4.A), the average travel speeds, averaged over the hour measured, should be determined for the peak hours of the day. The average speeds over a segment of highway may be determined using the test vehicle, license plate, or photography methods, as described in the *Traffic*

*Engineering Handbook.* Peak-period traffic volumes and average travel speeds can be fed into the Congestion Needs Assessment Model used by Regional Planning to assess vehicle hours of delay (VHD).

#### 5.2.3.5 Capacity Analysis Updates

Judgment must be used whenever a new software release becomes available or when revisions to the HCM are made, as to whether previously completed analyses should be reevaluated. While many factors may enter into this decision, the overriding consideration is whether it is likely that a new analysis will significantly change the design, investment, and/or environmental decisions. The final determination on whether to redo an analysis rests with the Regional Design Engineer. Refer to Section 5.2.4 for updating traffic data and traffic analyses.

Refer to Section 5.9.2 for guidance regarding intersection capacity and level of service analysis. Additionally, refer to the roundabout pages on the Department's Internet (<https://www.dot.ny.gov/main/roundabouts>) and IntraDOT sites for guidance and requirements on roundabout capacity and level of service analysis.

#### 5.2.3.6 Capacity Analysis Results

Refer to the [HDM Chapter 5 Internet page](#) for the traffic analysis report format. The report includes a summary of the methodology, the tabulated results, a summary of the results, and turning diagrams.

Tabulate for Existing, ETC and Design Year(s) for all peaks:

- LOS
- Delays
- Queues (from VISSIM or Sim Traffic but not from Synchro since Synchro will underestimate queues in oversaturated conditions)
- Travel speeds

Software files are to be named logically and placed in ProjectWise. Output reports are to be printed to pdf, named logically, and placed in ProjectWise.

#### 5.2.3.7 Exceptions to the Required Capacity Analysis Methodology

Proposals to use an analysis procedure other than the HCM or the Department-approved capacity software must be submitted to the Main Office for approval. Proposals for projects in preliminary planning, up through the project scoping stage, should be submitted to the Statewide Policy Bureau. Proposals for projects in either the preliminary design or detailed design stages should be submitted to the Design Quality Assurance Bureau.

There are cases when capacity and level of service are inadequate measures to document the traffic performance of an existing or proposed facility. These cases often involve complex geometric and/or signal control situations (e.g., Intelligent Transportation System/Advanced Traffic Management System); roadways or ramps which are oversaturated; or where the proximity of controls (e.g., signalized intersections) cause spillback, affecting nearby locations.

In these cases, use of queue analysis and/or traffic simulation models to estimate other traffic performance measures should be considered in addition to capacity and level of service. Contact the Design Services Bureau or the Statewide Policy Bureau for guidance in these situations. Also, note that the definition of Level of Service in simulation models such as Vissim is different than the HCM definition and therefore, the density output reported by the model should not be used to compute HCM LOS, but rather used as is.

#### 5.2.4 Updating Traffic Data and Capacity Analysis

Accurate design-year traffic data is needed to help evaluate the effectiveness of feasible alternatives and to produce the most cost-effective designs that achieve full expected service life. Desirably, current traffic data should be used. However, since regathering data and redoing analysis is costly and time consuming, it may be acceptable to use older data and analysis under certain conditions. Consider updating the capacity analysis, traffic diagrams, and design year traffic forecast (prior to distribution of the draft Design Approval Document, Design Approval, and PS&E) if any of the following factors have the potential to impact the proposed design:

- The estimated project completion date is postponed by more than 4 years (e.g., the ETC+20 design year is changed from 2025 to 2030).
- New development has or is expected to occur that will substantially affect the traffic analysis.
- Travel patterns have or are expected to change substantially.
- Project limits have been expanded or modified substantially.

Where volumes are low, LOS is A or B and the project is postponed or the limits have changed, updating the analysis is often not practical.

Traffic data, forecasts, and analyses, whether current or not, should be reviewed with the Regional Transportation Systems Operations Engineer. If updated information is being considered, consult with the Regional Planning & Program Manager and the Regional Transportation Systems Operations Engineer on the need for, and how to do, an update of the design-year traffic volumes, traffic diagrams, and capacity analysis.

If the older data will be used, consider spot checking current traffic patterns to determine if the older data projected to the current year is representative of current conditions. Average hourly speed data can be checked using the floating car method. Critical turning movements should be recounted, as necessary, to ensure adequate storage length, number of turn lanes, etc., is provided.

The rationale for the retention and use of older data needs to be documented in the Design Approval Document or, if design approval has already been obtained, in the permanent project files.

#### 5.2.5 Speed Studies

Speed studies provide an essential measure for evaluating highway geometry. The speed study results also serve as the basis for selecting a design speed within the acceptable range for the highway's functional class (refer to Section 2.7 of this manual). The Regional Transportation Systems Operations Group should be consulted on how to conduct these studies in order to obtain

statistically reliable results.

As an exception to a formal speed study, the Regional Transportation Systems Operations Engineer can select an off-peak 85<sup>th</sup> percentile speed equal to or above the regulatory speed based on their expertise and experience.

**Note:** The regulatory speed alone is generally not a reasonable indicator of the off-peak 85<sup>th</sup> percentile speed. Numerous studies (including FHWA's "Effects of Raising and Lower Speed Limits on Selected Roadway Sections," 1997) have shown that speed limits have only a very minor effect on operating speeds and cannot reliably be used to predict the operating speed.

#### 5.2.5.1 Speed Terminology

##### A. 85<sup>th</sup> Percentile Speed (Operating Speed)

The operating speed is a single speed that reflects the majority of motorists. Rather than use an average speed, which may only accommodate half the highway motorists, the Department and most transportation agencies use the internationally accepted off-peak 85<sup>th</sup> percentile speed to represent the operating speed. The 85<sup>th</sup> percentile speed is the operating speed that only 15% of the motorists exceed during off-peak hours.

##### B. Recommended Speed

The recommended speed is the maximum speed, under optimal conditions, considered appropriate for a particular location. The recommended speed should consider the alignment and sight distance. Other physical conditions, such as narrow lanes, roadside development, steep grades, etc., may also be considered.

The recommended speed based on the vertical sight distance should be determined from Appendix B of this chapter. The recommended speed based on the horizontal sight distance should be based on Section 5.7.2.4. The recommended speed based on the superelevation and radius should be determined by (in order of preference):

- Calculating the speed from the geometry and equation from Section 5.7.3 of this chapter.
- Using a ball bank indicator reading of 10° for horizontal alignment.
- Using Figures 5-8 and 5-9 of this chapter when horizontal alignment when the radius and superelevation are known.

**Note:** Each method will result in slightly different results.

##### C. Advisory Speed

The advisory speed is defined as the recommended speed rounded to the nearest 5 mph, but not more than the posted speed.

##### D. Regulatory or Legal Speed Limit

The Regulatory or Legal Speed Limit is the maximum speed along a highway segment allowed by local or state regulations. It may also be referred to as the posted speed when regulatory signs are posted. When regulatory signs are not posted, the speed limit is the statutory speed.

#### E. Statutory Speed Limit

The statutory speed limit is 55 mph as established by the NYS Vehicle and Traffic Law.

#### 5.2.5.2 Speed Study Methods

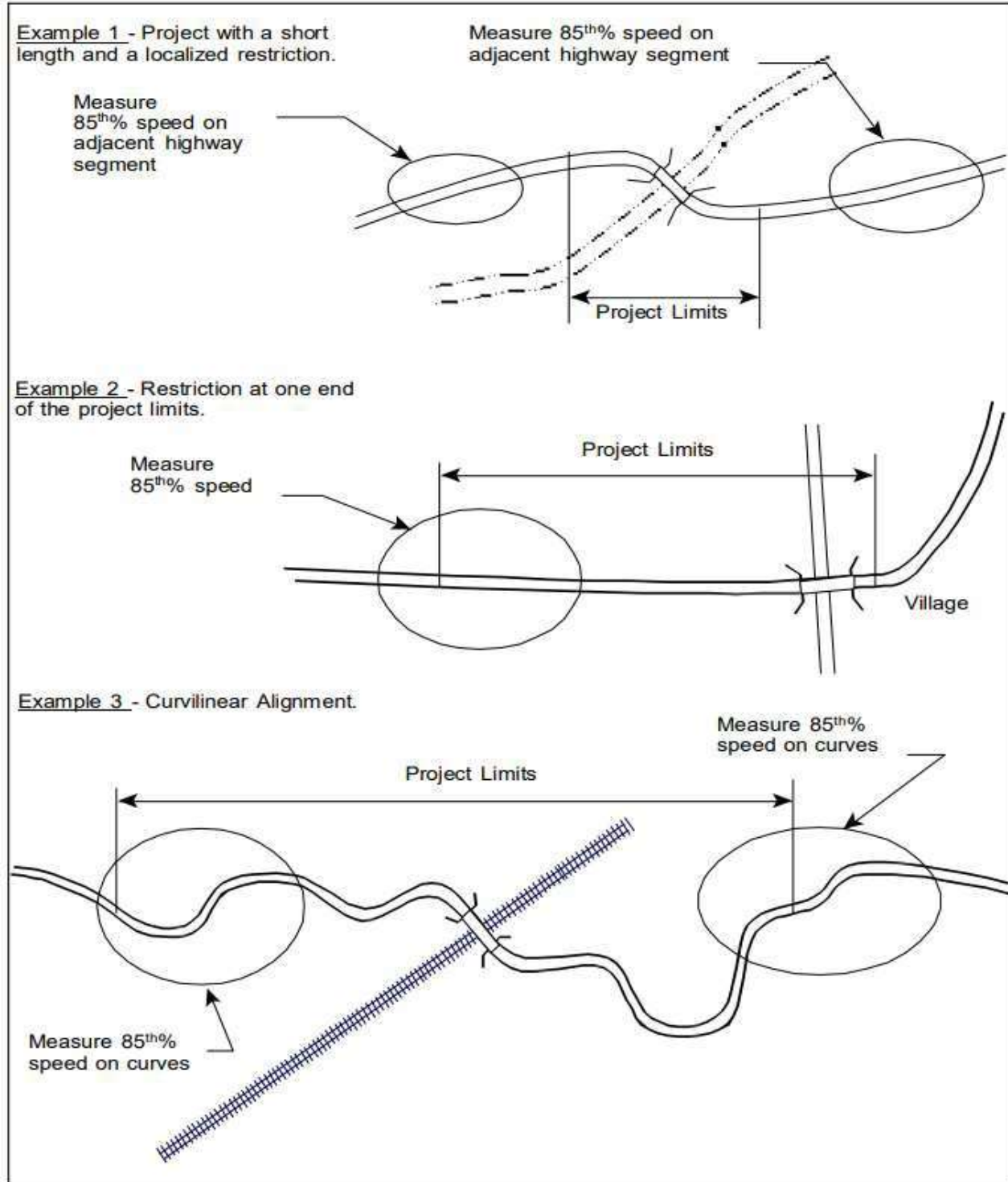
The existing operating speed can be determined or estimated during the off-peak hours by using (in order of preference):

1. Speed measuring devices.
2. A radar spot speed study of at least 30 vehicles (preferably 50 vehicles) that can be performed during off-peak periods. This is generally only practical for highways with 250 vpd or greater.
3. The data used to set a speed limit at the project site, if such data is still representative of current and anticipated operating conditions.
4. Test cars or following-car techniques during off-peak periods.
5. The statewide operating speed study for a similar facility. This information is available from the Highway Data Services Bureau in the Office Technical Services.

#### 5.2.5.3 Speed-Study Location

Select a speed-study location where motorists are not affected by localized nonstandard features or traffic control devices (e.g., stop signs, narrow bridges, sharp curves). The study may need to analyze both directions to ensure that it measures the highest speeds. Refer to Exhibit 5-2 for examples of how to locate the speed study.

## Exhibit 5-2 Speed Study Locations



NOT TO SCALE

### 5.3 CRASH ANALYSIS

Identifying the cause(s) of crashes will usually provide an insight into what corrective measures may be taken to minimize potential future crashes. Around 1,000 fatal crashes and nearly 200,000 injury crashes occur per year on New York state and local highways. Approximately 40 percent of the fatal crashes and 30 percent of the injury crashes occur on the State Highway System. The estimated average cost of a fatal crash in New York State is \$12,258,933 and the cost of a serious injury crash is \$689,924.

The purpose of a crash analysis is to identify crash patterns which may be correctable by studying and quantifying crashes within and immediately adjacent to the project limits. The analysis should then isolate and identify the causes of crash patterns and clusters, and identify potential countermeasures. The Regional Traffic Group can either perform the analysis or assist in its conduct and interpretation.

#### 5.3.1 Applicability

An Initial Crash Screening and either an Intermediate or Advanced Crash Analysis shall be performed on every highway and bridge project that offers an opportunity to address crash causes or severity as identified by the Department's CLEAR application (see below) or the Highway Safety Manual (HSM) formulas.

Examples of projects that will not likely benefit from a crash analysis include :

Element-Specific maintenance projects, such as sidewalk/ADA curb ramp repair, pavement marking contracts etc.

Bridge preventive maintenance projects, such as Element-Specific Cyclical Bridge Work, joint repair, bearing replacement, etc.

#### 5.3.2 Timing and Responsibility

A crash analysis and previous Highway Safety Investigations or Operation Studies can aid in the development and evaluation of project alternatives, and in determining the need for potential safety improvements. Therefore, a crash analysis and study review should be performed early in project scoping and documented in the Project Scoping Report and in the Design Approval Document (DAD).

Project developers, in conjunction with the Regional Traffic Group, are responsible for retrieving and analyzing crash data in accordance with this procedure and for incorporating scope appropriate crash countermeasures or other safety enhancements which address the identified crash patterns into each capital project. To achieve the Department's goal of continually improving highway safety for the public, effective crash countermeasures and enhancements should be considered for inclusion into its projects. Any identified mitigation measures which are not included should be considered for future project programming.

During the project development process, the crash analysis should be reviewed when the latest data used in the analysis is 5 or more years old OR if substantial changes have occurred at the project site that may affect crashes. These changes may include different traffic patterns or substantial volume changes; increased intensity or change in type of development (commercial, industrial, residential, etc.); new/different traffic control devices (signals, signs, markings, etc.); roadway feature changes,

etc. The crash analysis should be updated if a new crash pattern or cluster has appeared, and cost-effective mitigation measures should be recommended, as appropriate. Any revisions to the recommendations or proposed mitigation measures should be documented in either the Design Approval Document (prior to design approval) or a reevaluation statement if design approval has been obtained.

### **5.3.3 Crash Analysis Procedures**

The crash analysis steps and level of review depend on the project type and opportunities for practical improvements. The Department's Crash Location and Engineering Analysis Reporting (CLEAR) application "Site Investigation" can assist in determining the potential for safety improvement compared to other similar highway sections. CLEAR will calculate the "Potential for Safety Improvement" based on New York State Safety Performance Functions (SPF) and the CLEAR Facility Type. The Initial screening will also identify specific crash types that are overrepresented for the selected facility type. This information can be used to determine if the Intermediate or Advanced analysis procedures are appropriate. For details on the initial screening process and the CLEAR Site investigation procedures, refer to the New York State Highway Safety Investigations Manual (which is available on the Highway Safety and Traffic Operations webpage). The Regional Traffic engineer or designee should confirm the level of analysis required.

### **5.3.4 Highway Safety Investigation Report**

The Regional Traffic Group can provide a number of project supporting documents based on a safety investigation. Depending on the level of analysis performed, this may include summary crash statistics, collision diagrams, site reviews, safety recommendations, and alternatives analysis. These documents are not all required as part of a crash analysis for a capital project but the problems identified and proposed solutions should be discussed early in the project development process.



## 5.4 VACANT

## 5.5 RIGHT OF WAY (ROW)

The designer is responsible for determining the right of way needs necessary for the construction and maintenance of a proposed project. An assessment of right of way needs should be made during the project scoping stage. Specific right of way needs are determined after the various design alternatives have been identified and evaluated. Each alternative's potential impact upon the residents, environment, neighborhood, businesses, land use, and users should be examined and evaluated. The designer should coordinate with all involved program area groups to gain a varied perspective on the impacts of all alternatives. This should be done as early as possible and programmed into the project schedule.

The right of way processes described in this section are applicable to Design Build projects. Refer to the NYSDOT *ROW Instruction Manual* and the NYSDOT [Design Build Procedure Manual](#) for additional information.

### 5.5.1 Abstract Request Maps

Once properties which are affected by the various alternatives being considered are identified, the designer should submit the proposed right of way limits to the Right of Way Mapping Group (i.e., consultant or Regional). These proposed right of way limits will be outlined on base mapping, as described in Chapter 3 of the [Right of Way Mapping Procedure Manual](#), to create an Abstract Request Map (ARM). The Right of Way Mapping Group sends the ARM to Regional Real Estate for forwarding to the Department of Law (DOL).

An ARM is prepared to obtain the necessary title data for the properties that may be acquired by the project. It provides the DOL with a means of identifying the properties for which title data is required. Supplemental ARMs should be submitted whenever changes in a project's work limits occur which will affect which properties are being acquired. DOL requires title data before they can provide title certification of most right of way acquisitions. Title certification is required by Real Estate (i.e., Regional and Main Office) before compensation offers can be extended to land owners.

The size of the ARM will vary depending upon whether it includes just the preferred alternative, or all of the feasible alternatives. The decision on how many alternatives to include should be based upon the size of the project, the amount of time needed to research the title data, the cost to the Department for requesting additional title searches, and the complexity of the title data along the project's corridor. The designer should consult with the Regional Real Estate Group and review the project schedule to determine when this information needs to be available. ARMs should generally be submitted to the Real Estate Division a minimum of 12 to 24 months prior to the PS&E for the project, depending upon the amount and complexity of the title data requested.

A "Table of Temporary Reference Numbers" (TRNs) is generated as part of the ARM. This table lists all proposed acquisitions from properties that could be affected by the project. (Refer to Chapter 3 of the [Right of Way Mapping Procedure Manual](#) for more information on TRNs). This table will be updated and further expanded during later stages of the project to tabulate the anticipated right of way acquisitions.

### 5.5.2 Design Approval Document (DAD) and Preliminary Plans

The designer should provide information about the magnitude, types, and overall cost of preliminary right of way for each of the feasible alternatives in the Design Approval Document (DAD), for both federally and state-funded projects. This information is provided to the public and other evaluators of the project to assist them in determining the magnitude of the right of way acquisitions and the proposed limit to which the acquisitions extend into private and public properties for each feasible alternative.

All feasible alternatives which involve right of way acquisitions should be shown on the preliminary plans in the appendix of the DAD. These preliminary plans should include, along with existing and proposed highway alignments, the schematic delineation of the approximate limits of existing and proposed right of way (Refer to Appendix 7 of the [Project Development Manual](#)).

**Note:** Right of Way Plans, for use in a separate right of way approval process, are not required as part of the documentation of right of way required for a project. The contract plans will provide the Department's documentation of what right of way was determined to be necessary to be acquired for a project.

In addition, the DAD should also include (in the appendix) for each feasible alternative, a "Table of Anticipated ROW Acquisitions" which lists all property owners from whom right of way is anticipated to be acquired (Ref. [Project Development Manual](#), Chapter 4). A tabulated list is not required for a project which does not have ROW acquisitions.

The "Table of Anticipated ROW Acquisitions" should include the following information for each property from which a ROW Acquisition will be appropriated:

- Land owner's name
- Type of acquisition
- Estimated ROW area to be acquired

This summary of ROW information will be utilized by private land owners, municipalities, and FHWA (if federally funded) to evaluate the magnitude and limit of the ROW acquisitions as part of the project review process. This information will also be used by the Regional Real Estate Group (upon receipt of the final Design Approval Document) to obtain Acquisition Phase Authorization.

### 5.5.3 Right of Way Determination

The design should include proposed ROW lines on the working plans which encompass the areas required to access, construct, and maintain the proposed facility. The designer shall allow room beyond the construction limits (toe or top of slope) for construction equipment and for future maintenance operations, such as mowing or cleaning ditch lines. The type of acquisitions should be determined by the use for which the land is taken, the party for whom the access will be provided, and whether the need for the land will continue after the construction contract is completed. If no work extends beyond the existing highway boundary, there is no need to acquire additional ROW. General guidance for width of right of way to be acquired beyond the construction limit includes:

- A distance of approximately 10' (3 m) is desirable when it can be obtained with little extra cost or impact to the adjacent property, such as in rural areas with no nearby dwellings.

- A minimum of 1.5' to 5' (0.5 m - 1.5 m) should be used where right of way costs are more expensive or impacts are more significant, such as on front lawns of houses or in commercial areas with limited building setbacks.
- A minimum of 1' (0.3 m) should be considered in urban settings where buildings are set close to the roadway, or outside of sidewalks which are separated or detached from the highways by a wide utility strip or grass area.

In addition to cost, consideration should be given to visual aesthetics, maintenance activities, roadside safety (i.e., clear zones and clear areas), regional guidance such as "Guidelines for the Adirondack Park", future development (e.g., zoning setbacks), accommodation of utilities, drainage design, soil erosion and sediment control during construction, and disruption to adjacent property owners, when determining taking lines. Discussion with all involved functional units, municipalities, utility companies, and regulatory agencies should occur early in the design process to ensure all impacts are fully evaluated.

The reestablishment of driveways, private sidewalks, and other approaches to private lands should be accomplished by use of releases. This work shall only include what is necessary to reconnect a privately owned approach to the adjoining highway. This work should not include any construction activities that are critical to the successful completion of the project. Therefore, work done within a release should not include grading for roadway support, installation of highway drainage (as opposed to those driveway culverts which do not form a part of the highway drainage system), municipal utility lines or structures, or construction of public sidewalks. Refer to Section 5.5.6.6 for additional guidance regarding reestablishment of approaches to private lands.

Taking lines should generally avoid frequent angle points. When angle points in the taking lines are necessary, they should generally be kept a reasonable distance ( 10' (3 m) minimum) from property lines which are transverse to the roadway, to avoid being mistaken for property line corners between adjacent owners.

Refer to Section 5.5.6 for additional guidance regarding types of right of way and access.

#### **5.5.4 Taking Line Review Meeting**

Once the preferred alternative is chosen for a project and the initial right of way taking lines are detailed, the designer is to schedule a meeting (commonly referred to as a "Taking Line Review") to discuss the limits and types of the proposed right of way acquisitions, any concerns, the project schedule, and make final determinations regarding the size and type of acquisition(s) to be mapped. On large projects, it may be advisable to break the project into segments and schedule separate meetings to discuss each segment. The taking line review meeting is to include the project designers, consultant manager (for consultant-designed projects), and Regional representatives from:

- Real Estate
- Survey
- Landscape Architecture
- Environmental Services
- Right of Way Mapping

Representatives from Regional Construction, Maintenance, Transportation System Operations, and Program & Project Management are to be invited to attend the meeting. If taking line changes become necessary after the meeting, the group should be reconvened to discuss the changes.

For a taking line review meeting, the designer should portray the following information on project base mapping so that it is easily understood by the attendees. Colored lines or colored shading may be used to improve clarity.

- Baselines and center lines.
- Proposed construction work limits such as toes and tops of slopes or safety-related clear areas.
- Anticipated construction operations and stages, traffic control plans, erosion and sediment control plans.
- Proposed structures such as bridges with wingwalls, buildings, sidewalks, retaining walls, and sign and lighting structures.
- Existing private underground services such as utility lines, wells, septic systems, and storage tanks (especially when the site is known as a former gasoline station location).
- Approximate boundary of contaminated soils, if known.
- Existing and proposed access control delineated and labeled.
- Existing and proposed aboveground and underground private and municipal utilities (e.g., fire hydrants, underground utility lines & structures, utility poles, signal poles, pull boxes).
- Proposed drainage facilities, including piping, underground structures, headwalls, open ditch lines with the direction of flow indicated, and stormwater management facilities.
- Existing highway boundary lines and proposed right of way acquisitions.
- Types of acquisitions indicated and labeled with the purpose of each easement.
- The limit of work on all side roads and driveways.
- Any building acquisitions and all structure encroachments into the ROW.
- The separate identification of all properties from which ROW is being acquired but were not identified, or were identified but are no longer needed, on the Abstract Request Map.

In addition to the project mapping, the designer is to provide cross sections with the proposed construction work limits and right of way limits shown for reference during the Taking Line Review.

### **5.5.5 Design Phases V-VI, Final Design Stage**

The contract plans will document what right of way acquisitions were determined to be necessary to construct and maintain the project with the ROW acquisition maps serving as documentation of the actual right of way acquisition. Thus, the contract plans should include an accurate representation of all the right of way acquisition maps which demonstrate the proper ties which are to be appropriated for that project. To provide this documentation, the contract plans should include a graphical presentation of all acquisitions on the general plans. Separate "Acquisition Plans" should be prepared for projects in which the general plans would otherwise be too congested to clearly portray both the construction improvements and the right of way acquisitions on the same plan sheets. In addition to the general plans or acquisition plans, a "Table of Right of Way Acquisitions" shall be prepared. This table is expanded from the Design Approval Document "Table of Anticipated Right of Way Acquisitions." To keep this information current and inclusive of all changes that occur during design refinement, frequent communication between the designer and ROW Mapping Group is necessary, so that all changes can be reflected on both the contract plans and the ROW Acquisition Maps. The designer should also contact other groups when changes affect their interests.

Design changes in areas of land acquisitions need to be communicated to the Regional Real Estate Group during final design, so they are aware of potential impacts to adjacent landowners. Regional Real Estate meets with each of the affected landowners along a project to describe the type and size of the acquisition that the State is appropriating from their property. These discussions include how the project will affect the topography, structures, and landscaping of a property. Therefore, Regional Real Estate needs to be kept abreast of construction impacts and any changes to those impacts on adjacent land parcels and receive periodic updates of general highway plans, profiles, and cross sections.

Design changes in areas of land acquisitions that impact environmental issues (e.g., historic, parkland, wetland) need to be communicated to the Regional Landscape and Environmental Section during final design, so that they are aware of any changes to permits or mitigation needs.

The "Table of Right of Way Acquisitions" shall include:

- Reputed owner's name.
- Map and parcel numbers.
- Types of acquisitions. PEs and TEs shall include the purpose of the easement.
- ROW area to be acquired.
- State highway number (on projects which include more than one state highway).
- 

A copy of this table should be forwarded from Design to Regional Real Estate at least two weeks prior to PS&E, for Real Estate's use as a checklist to determine if all acquisitions have been processed, and for obtaining ROW Certification. This list in the plans will provide a tabulation of all ROW acquisitions for use by the Contractor and EIC, as well as for historic reference of what ROW was acquired for that specific project.

### 5.5.6 Types of Right of Way and Access

Right of way is usually acquired by appropriation. Appropriation is the acquisition of property by the government through the right of eminent domain. **The amount of right of way acquired shall be limited to only what is necessary to access, construct, or maintain a highway and its designed features and appurtenances.**

The different types of right of way acquisitions and access are described in Sections 5.5.6.1 through 5.5.6.9.

#### 5.5.6.1 Fee with full access - Fee (W/A)

##### A. Definition

Acquiring absolute right, title, or estate to a parcel of land for use by the state for purposes related to highways or other transportation related facilities.

##### B. Types of Use

Fee with full access is used for the construction or maintenance of roadway pavements, structures, appurtenances, and their supporting foundations. Fee acquisitions should be appropriated to include all permanent structures which are part of the highway infrastructure, but are not within the existing highway right of way. These should include wing walls, headwalls, guide rail, sign structures, signal equipment, public sidewalks, drainage structures, and retaining walls used to support the highway. In addition, fee acquisitions should be appropriated to include all highway elements which are necessary to support or protect the integrity of the highway (e.g., roadside ditches or side slope protection installations) or to mitigate environmental impacts associated with the proposed project (e.g., the acquisition of land for wetland creation).

In certain situations, such as in highly developed commercial areas, a fee acquisition may reduce a commercial property to below a standard size lot required by local zoning or reduce a building setback below the minimum requirement. These two situations could cause undue hardship on the owner should they end up with a substandard lot. The implications of a fee acquisition in these cases far outweighs the cost of the land, therefore, a Permanent Easement acquisition may be more appropriate.

Acquisition of ROW should be avoided around areas which include private wells and septic systems, underground tanks, private drainage systems which collect, transport, or discharge storm water from a private property, and retaining walls which support private property embankments. Appropriation of these facilities will implicate the state for their maintenance responsibility or liability in the future.

### 5.5.6.2 Fee without access - Fee (W/OA)

#### A. Definition

Same definition as Fee W/A, except that the remainder parcel of the abutting owner is denied direct access across the fee parcel to the public highway. Fees W/OA that are purchased to limit access are usually acquired at a minimum width of approximately 1 ft, but can include the entire acquired parcel.

#### B. Types of Use

Fee without access is used to control (deny) access onto “limited access” types of highways such as interstates or parkways, or to control access in highly congested or crash prone portion of highways. Fees W/OA may also include all of the types of use listed under Fee W/A.

### 5.5.6.3 Permanent Easement (PE)

#### A. Definition

Permanent easement is the acquisition of certain rights and interest to use or control a property for a designated purpose. In most cases, the property owner retains the use of the property for other functions which do not interfere with the purpose of the easement.

#### B. Types of Use

Permanent easements provide for the limited use of private property which is necessary for highway purposes. The PE Map must describe the specific right that is being acquired and for what purpose. Examples of this use are:

- Drainage - To control the direction and maintain the flow of storm water. This may include minor underground drainage lines which collect storm water from low areas adjacent to the highway, or discharge storm water to naturally occurring watercourses which lie adjacent and downstream of the highway. (See also, discussion of rights of entry in Section 5.5.6.8.)
- Sight Distance - To allow for clearing and maintaining of a critical sight distance area.
- Slopes - To maintain the stability of a side slope along a highway which does not support a critical highway element, (e.g., large back slope out from a ditch line).
- Viewsheds - To allow for the conservation and development of roadside view sheds and natural features.
- Maintenance Operations - To allow maintenance crews access to highway structures or other appurtenances for the intent of maintaining their integrity or purpose.

- Bank Stabilization - Banks of streams or rivers which are susceptible to erosion near a highway may require stabilization efforts to be maintained.
- Commercial Access Control - Along highways, where commercial access control is desired, a PE can be acquired for highway purposes to allow for construction of roadside traffic control elements (PEs can be used in these instances, as a last resort, where minimum commercial lot sizes, setbacks, or green spaces would be reduced and property value would be significantly compromised by a fee taking).
- Clear zones and clear areas - To create and maintain clear zones and clear areas.
- Railroad properties - For construction of any permanent highway bridge structures across an operating railroad property, in most situations a PE is acquired from the applicable owner. In some situations, specific railroad companies have not agreed to PEs, but have provided permits to allow for the construction work and future maintenance.
- Contaminated soil - To acquire the rights of access and/or use a piece of real estate without owning the underlying fee and contamination liability. Refer to Section 5.5.6.9 A for additional guidance.
- Snow fence - to allow for the construction and maintenance of permanent snow fence installations.

#### 5.5.6.4 Temporary Easement (TE)

##### A. Definition

Temporary easements acquire the use or control of a piece of property for specific use(s) during a construction project, for a set or limited duration of time (usually the length of the construction contract). The owner is compensated for their inconvenience, loss of value, or loss of access on the TE.

##### B. Types of Use

Temporary easements should be used for work which is essential to the proper, timely, and safe completion of the project, but not for work which restores private access to a highway. Examples of temporary access are:

- For construction and removal of a temporary detour or onsite diversion, including bridges.
- For construction and removal of temporary pedestrian bridges or walkways.
- To demolish or raze a structure on a property where the state has taken title to the structure, but does not own the underlying property.
- To slightly modify the land features or grade characteristics of an adjacent property that improves the safe use or integrity of the highway while not affecting the existing land use.
- To allow access for specialty construction equipment such as pile drivers or cranes.



- For stream realignment including the excavation or clearing of a new streambed or backfilling and seeding a former streambed both of which reside on the same owner's property.
- For use by the contractor for the temporary storage of equipment or materials used on the construction project (if deemed beneficial to the Department), or for temporary access by the contractor across private property.
- To install and maintain temporary soil erosion control measures.

#### 5.5.6.5 Temporary Occupancy (TO)

##### A. Definition

A temporary occupancy maps a specific area of private property, which under Section 404 of the Eminent Domain Procedure Law and Section 30 Subdivision 17 of the Highway Law allows the state or its designees (contractors) to enter upon for project -related business. A TO allows for compensation to the landowner up to \$2500 for loss of use or damage to their property during construction activities. However, since the land is not appropriated, some situations have arisen during which construction work has been delayed by the landowner, or the assessed damages have exceeded the \$2500 limit. In some cases, the TOs have been reprocessed as TEs; duplicating much of the effort.

##### B. Types of Use

Temporary occupancies are discouraged and shall only be processed with the concurrence of the Regional Real Estate Group. TOs are only to be used in isolated situations where there is a definite advantage to the state, and then, only used on areas which are noncritical to the completion of the project.

#### 5.5.6.6 Reestablishment of Approaches to Private Lands

Section 54a of the Highway Law, Reestablishment of Approaches to Private Lands, allows the Department to reestablish existing entrances, approaches, or driveways to meet the new highway grade. Entrances, approaches, and driveways have been interpreted to include driveway, curbs, sidewalks, stairs, etc. Before the contractor is allowed to make any adjustments outside the State's ROW, a release from the property owner must be obtained as discussed in Section 107-14 of the *Contract Administration Manual, MURK Part 1A*. The release provided in Section 107-14 should be used. A release is a nonbinding agreement (without compensation) between a landowner and the state to allow for the reconnection of a private or commercial access to a highway. No TE or TO maps are used for this purpose, and no compensation is paid to the land owner since the reconnection is for their benefit. No project-related work should be included under this release that, if the owner were to deny access, would prohibit the contractor from completing an essential element of a project.

Refer to Appendix A of this chapter for the Driveway Design Policy

#### 5.5.6.7 Planting Trees and Shrubs Along State Highways

Section 19 of the Highway Law, Planting Trees and Shrubs Along State Highways, allows the Department to plant trees and shrubs on private property with the consent of the property owner. If plantings on private property are to be specified, form HC91, illustrated in Section 107-14 of the *Contract Administration Manual, MURK Part 1A* must be used to obtain the property owner's permission before the work can begin.

#### 5.5.6.8 Rights of Entry

The rights of entry discussed in Sections 5.5.6.8 A and B should be used with caution. If exercised inappropriately or without proper cause, it could lead to claims filed by the adjacent land owners. These rights do provide for access in situations where the integrity of the roadway is threatened or the safety to the public users of the highway could be in jeopardy. For any work in a stream or creek which is performed outside the existing highway boundary and alters the channel location, flow characteristics or the underlying land use, should be accomplished within ROW acquired by appropriation. This appropriation should compensate the adjoining land owner for any change in the riparian rights they had prior to this work.

##### A. Section 45 of the NYS Highway Law

This section states that DOT employees or contractors working for the state can enter upon lands adjacent to a state highway or which contain a stream or creek to:

- Open, maintain, or construct an existing ditch or drain for the free passage of water for drainage of such highway.
- Construct, reconstruct, or maintain drainage channels in order to keep the waters of such streams or creeks within their proper channel and prevent their encroachment upon state highways or bridges.
- Remove or change position of a fence or other obstruction, which in DOT's judgment prevents the free flow of water under or through a state bridge or culvert.
- Remove private fences or obstructions which cause snow to drift in and upon a state highway, or to construct or remove temporary snowfences which prevent the drifting of snow in or upon a state highway.
- Inspect, remove, or prune trees which in DOT's judgment constitute a danger to the users of the adjacent highway.

##### B. Section 404 of the Eminent Domain Procedure Law (EDPL) and Section 30, Subdivision 17 of the Highway Law

These sections of the law allow for DOT employees or contractors working for the state to enter upon private land, prior to acquiring any real property, to engage in work connected with a proposed public project. This right of entry shall be for the purpose of making surveys, test pits and borings, or other investigations needed for the project. The state's representatives shall be responsible to notify the private land owners, by mail, prior to entry upon the land. The state shall also be liable to the landowner for any damages caused by or as a result of entry.

#### 5.5.6.9 Special Considerations

##### A. Contaminated Soil

Acquisitions along properties which have in the past or still do contain commercially sold or used hazardous materials (including petroleum based) should be investigated for possible contamination of the soil on the site. Hazardous waste and contaminated materials procedures described in the [Environmental Manual](#) should be followed and investigations coordinated with the Regional Environmental Contact. When acquisitions are deemed necessary on properties which have been determined as contaminated, care should be used in determining the limits and types of acquisitions due to possible legal implications. Therefore, the following guidance is provided to assist in these determinations:

- Outer limits of the soil contamination in areas of possible ROW acquisitions should be determined as closely as present technology allows.
- Existing sources of the contamination should be investigated, and appropriate action taken to prevent further contamination. If any possible sources of contamination are located within a proposed acquisition, (such as underground petroleum storage tanks or piping system which leaks), it shall either be avoided, or removed and appropriately disposed of. This removal shall be performed under the use of a Temporary Easement.
- Acquisitions of hazardous waste contaminated soils which are also likely necessary, (other than petroleum-based contamination) shall be acquired as Permanent Easements. This action avoids acquiring the underlying fee title and the possible contamination liability. Final determinations of these type of acquisitions should be coordinated with the Department's Office of Legal Affairs.
- Acquisitions of petroleum-based contaminated soils, (which do not include any remaining sources of the contamination) are regulated by different federal and state statutes than other hazardous wastes, and thereby have different liabilities associated with them. Thus, fee acquisition of petroleum contaminated soils may be permitted, if the Department does not acquire any part of the system from which the release is believed to have occurred. If uncertain of the potential implications of a specific acquisition, seek legal counsel from the Department's Office of Legal Affairs.

##### B. Utility Easements

Utilities acquire easements from private land owners for the purpose of locating their facilities across private property. These easements are generally affected when a proposed project requires acquisition of some or all of the land rights of the same property that the easement is located upon. The two ways the Utility easement may be affected are as follows:

- An acquisition in which the Utility is required to relocate their facility - In this case, all project-related relocations of utility facilities located on private property will be reimbursed from construction funds pursuant to Section 10 (24-b) of the Highway

Law, and [Chapter 13](#) of this manual. By reimbursement for this relocation, all existing Utility easements located within proposed acquisitions shall be assumed to be compensated for and extinguished.

- An acquisition in which the Utility is not required to relocate their facility - In this case, the Utility retains their easement rights that they held prior to the proposed acquisition. The proposed acquisition is thus made "subject to" the rights previously held by that Utility.

### 5.5.7 Encroachments

Encroachments exist on many highway rights of way. Generally, the owners should be requested by Regional Transportation Maintenance to remove these encroachments. However, an encroachment may be allowed to remain if it can be shown that the structure in no way impairs or interferes with the free and safe flow of traffic on the highway. Encroachments are allowed to occur when a "Use and Occupancy Permit" has been granted, however, FHWA must also approve an encroachment that remains on a project requiring FHWA's design approval. The designer, in consultation with the other program area groups, may recommend to the Regional Director that the encroachment remain. If approval is granted, the Real Estate Group is responsible for managing the encroachment.

### 5.5.8 Excess Right of Way

Excess right of way is defined as existing transportation property beyond that which is sufficient to ensure safe, efficient operation of the highway as it exists and as it will exist in the foreseeable future. Changes in the highway alignment often result in an excess of right of way, or an existing excess is noticed in the design process. Excess right of way is established on a project specific basis. When determining excess right of way, the designer must consider the following:

- Probability of the need for future improvement (check the present volume/capacity ratio, level of service, crash rate, etc.).
- Horizontal sight distance.
- Adequate clear zone widths.
- Surface and subsurface drainage.
- Snow storage.
- Pedestrian facilities.
- Bicycle facilities.
- Bus turnouts.
- Traffic control devices.
- Utilities.
- Access control.
- Effects on property owners.
- Wetland mitigation.
- Preservation of views and aesthetics.

The Real Estate Group has responsibility for the disposal of excess right of way. See M.A.P. 7.8-5-1 Disposal of Surplus Real Estate and A02-5-29 Excess Property Identification for details of the procedure.

### 5.5.9 ROW Markers

Right of Way (ROW) Markers along a highway delineate the right of way:

- To assist adjacent land owners in the identification of the limits of the highway boundary adjacent to their property.
- To assist maintenance crews in determining the limits of the highway which they are maintaining.
- To monument the limits of the right of way, which provides secondary control for future reestablishment of the highway boundary.

#### 5.5.9.1 Where and When to use ROW Markers

ROW Markers are intended to delineate the boundary between the highway and private property, and mark any changes in the direction of that boundary line. All new right of way limits should be monumented as part of their associated construction projects. ROW Markers should be installed at all angle points along the proposed or new right of way boundary.

ROW Markers are not intended to monument the property lines between private properties. Therefore, no markers should be placed at property lines, except in unavoidable situations where an acquisition has to end at a property line. Recommendations on where and when to place ROW Markers should be reviewed by the Regional Land Surveyor.

#### 5.5.9.2 Which ROW Markers to Use

The Department uses concrete and steel pin and cap ROW markers. Refer to the Department's 625 series standard sheets for the ROW marker details. The following factors need to be considered when choosing between the various types of ROW markers. Proposed ROW Marker locations and types should be reviewed by the Regional Land Surveyor.

- Safety - Consider whether pedestrians, bicyclists, land owners, or vehicular traffic could be exposed to a hazard by placing a high or low concrete marker adjacent to, or within a sidewalk, public path, lawn area, or driveway.
- Aesthetics - Consideration should be given to the visual impact of ROW markers on a project area. For example, the placement of a line of high concrete markers (fence post look) may be visually intrusive to the project area. In situations where markers will be visually evident and could potentially create a less than desirable effect on the area's aesthetics, use of low or steel pin markers should be considered. Any use of concrete markers on or adjacent to historically significant or contributing properties, should be coordinated with the regional contact for historic and cultural resources.
- Land Use - Consider the present or anticipated land use for the adjacent properties. It is not desirable to install concrete markers in existing or proposed parking lots, driveways or sidewalks, maintained lawn areas, or on parklands. In contrast, it is advisable to consider the use of either high or low concrete markers near cultivated fields (since they need to be seen to be avoided), and use high concrete markers in

unimproved areas that have heavy underbrush, wetlands, or include standing water, to simplify their rediscovery in the future. Low concrete or steel pin markers are appropriate along interstates which also have fencing to delineate the right of way limits. Low concrete or steel pin markers should be used on or near commercial properties, depending on proximity to walks or driveways, and the resulting landscaped appearance after installation.

- Ground Conditions - Consider the types of ground materials or the underground utilities where markers are to be set. Rock outcroppings may necessitate the use of steel pin markers (by drilling and grouting), and high water table or unstable soils may warrant concrete markers to ensure their stability. While underground utilities warrant care on the contractor's part during installation, the designer may need to include a special note to establish the depth that a marker is to be set over utilities or pipes.

## 5.6 ECONOMIC ANALYSIS

The objective of economic analysis is to help select the most efficient transportation project or plan that minimizes the use of valuable resources (money, land, time, materials, manpower, etc.). A variety of methods exist for selecting the most cost-beneficial projects or for selecting a superior alternative from among a group of proposals. The following are brief descriptions of four common methods used by the Department, with simple examples of each. Exhibit 5-5 offers a reference to determine which method is usually used for various project types, and the functional unit generally responsible for the analysis.

**Exhibit 5-5 Economic Analysis Problem Types, Analysis Methods, Guidance in Force, and Contact for More Information**

Analysis Type	Method	Guidance	Contact
Bridge Rehabilitation vs. Replacement	Life Cycle Cost	<a href="#">Bridge Manual</a> Section 19	Office of Structures
Bridge Abandonment vs. Preservation Decisions	User Benefits Agency costs	Economic Analysis Worksheet for Bridges	Statewide Policy Bureau
Pavement Rehabilitation Alternatives	Life Cycle Cost	<a href="#">Comprehensive Pavement Design Manual</a> Chapter 3	Materials Bureau
Crash Reduction Treatments	Safety B/C Ratio (Safety Benefits / Project Costs)	Highway Safety Improvement Program Procedures and Techniques Manual, a.k.a. the 'Red Book'.	Office of Traffic Safety & Mobility
Mobility Betterment	Incremental B/C Ratios	Highway User Cost Accounting Package	Statewide Policy Bureau
Social, Economic & Environmental Impacts	B/C Ratios	<a href="#">Environmental Manual</a>	Environmental Science Bureau

### Methods

1. Present Worth Method - When two or more alternatives are capable of performing the same functions, the superior alternative will have the largest present worth when determining the present worth of benefits, and the smallest present worth when determining the present worth of costs. All alternatives must have the same lives and be mutually exclusive. The present worth for a single future benefit is calculated from the equation:

$$P_s = F(1 + i)^{-n}$$

The present worth for uniform annual benefits is calculated from the equation:

$$P_u = A \frac{(1 + i)^n - 1}{i(1 + i)^n}$$

Where:

- P = Present Worth
- F = Future Benefit
- A = Benefits per period (usually annual)
- i = Interest rate per period (usually annual)
- n = Number of compounding periods

The present worth of the net benefit of each alternative should be calculated by subtracting present alternative costs from present worth benefits.

**Example:** Given two projects, A and B,  $i = 5\%$ . A costs \$10,000 today and has a single future benefit of \$11,500 two years in the future. B costs \$8,000 today and has benefits of \$4,500 in each of the next two years.

$$\text{Present worth of the net benefit (Alt. A)} = -\$10,000 + \$11,500 (1 + .05)^{-2} = \$431$$

$$\text{Present worth of the net benefit (Alt. B)} = -\$8,000 + \$4,500 \times \frac{(1 + .05)^2 - 1}{0.05 (1 + .05)^2} = \$367$$

Alternative A is superior, since the present worth of the net benefit for Alt. A is higher than the present worth of the net benefits of Alt. B.

2. Equivalent Uniform Annual Cost (EUAC) - (also called Annual Return Method and Capital Recovery Method). The EUAC method assumes that each alternative will be replaced by an identical twin at the end of its useful life. The alternatives must be mutually exclusive and infinitely renewed up to the duration of the longest-lived alternative. The annual cost is given by the equation:

$$A = P \frac{i(1 + i)^n}{(1 + i)^n - 1}$$

Where:

- A = Annual Cost
- P = Present Cost
- i = Annual interest rate
- n = Number of years



**Example:** Given two highway improvement projects and the following information, determine which is superior over a 30-year period at an annual rate of 4%.

	<u>A</u>	<u>B</u>
<u>Type</u>	<u>Rehabilitation</u>	<u>Pavement Resurfacing</u>
Life	30 years	10 years
Cost	\$1,800,000	\$450,000
Maintenance	\$5,000/year	\$20,000/year

$$EUAC(A) = \$1,800,000 \times \frac{0.04(1+0.04)^{30}}{(1+0.04)^{30} - 1} + \$5000 = \$109,040$$

$$EUAC(B) = \$450,000 \times \frac{0.04(1+0.04)^{10}}{(1+0.04)^{10} - 1} + \$20,000 = \$75,485$$

Alternative B is superior, since its annual cost of operation is the lowest. It is assumed that three pavement resurfacing projects each with a life span of 10 years and cost of \$450,000 will be built to span the 30-year period.

3. Capitalized Cost Method - There are times when a series of equivalent uniform annual costs (EUAC) will start at some future date and must be combined with lump sum payments in other years. This is accomplished by converting the EUAC to a capitalization amount in the year the annual payments start. The capitalization amount is the amount of money that when invested today at the effective interest rate would give an annual return equal to the annual payments. It can be determined by the following equation:

$$CA = \frac{EUAC}{i}$$

Where: CA = Capitalization amount in year annual payments start  
 EUAC = Equivalent Uniform Annual Cost  
 i = Effective interest rate

The Capitalization amount in some future year can be converted to a Present Cost in a similar manner as the present worth is calculated above.

**Example:** A decision must be made whether to spend \$500,000 on a bridge rehabilitation project now or to do nothing and replace the bridge in the future at a cost of \$1M. Without the rehabilitation, the bridge would last ten years before replacement. The rehabilitation would add 15 years to the life of the bridge and, thus, the bridge would require replacement in 25 years. The effective interest rate is 4 percent and a new bridge life is 50 years. For perpetual bridge replacement:

$$EUAC = \$1,000,000 \times \frac{0.04(1+.04)^{50}}{(1+.04)^{50} - 1} = \$46,500$$

$$CA = \frac{\$46,500}{0.04} = \$1,164,000$$

#### Bridge Rehabilitation

1.	Present Cost of Rehabilitation	\$500,000
2.	Present Cost of Capitalization Amount occurring in 25 years: $P = \$1,164,000 (1+.04)^{-25}$	<u>\$437,000</u>
3.	Total Present Cost of this Alternative	\$937,000

#### Delay Rehab and Replace in Ten Years

Present Cost of Capitalization Amount in 10 years	
$P = \$1,164,000 (1+.04)^{-10}$	\$786,000

The analysis shows that delaying the rehabilitation and replacing the bridge in ten years will have the lowest life-cycle cost.

4. Benefit-Cost Ratio Method - To determine the B/C Ratio, the present worth of all benefits is divided by the total present worth of all costs. The project is usually considered acceptable if the B/C Ratio exceeds 1.

When the Benefit/Cost Ratio is used, disbursements by the initiators, or sponsors, are costs. Disbursements by the users of the project are known as disbenefits. It is often difficult to determine whether a cash flow is a cost or a disbenefit. The numerical result can be considerably different, since if it is disbenefit, it is subtracted from the numerator, and if it is a cost, it is added to the denominator.

**Example:** Given a public works project with:

Estimated benefits	=	\$ 1,600,000
Present costs	=	\$ 650,000
Additional cash flow item(s)	=	\$ 200,000

$$\text{Assume additional item(s) is a cost: } B/C = \frac{\$1,600,000}{\$650,000 + \$200,000} = 1.88$$

$$\text{Assume additional item(s) is a disbenefit: } B/C = \frac{\$1,600,000 - \$200,000}{\$650,000} = 2.15$$

For this reason, the B/C method should not be used to rank competing alternatives unless an incremental analysis is used. The optimum alternative may not necessarily be the one with the greater B/C. In order to do an incremental analysis, first determine that the B/C is

greater than one for each alternative. Then, for each possible pair of alternatives, calculate the ratio:

$$\frac{B_2 - B_1}{C_2 - C_1}$$

If the ratio exceeds 1, alternative 2 is superior to alternative 1. Otherwise, alternative 1 is superior.

**Example:** Given two highway improvements with the following information:

<u>Condition</u>	<u>Annual Road User Costs</u>	<u>Annual Highway Costs</u>	
Existing Roads	\$ 5,730	\$ 46	
Plan 1	4,760	234	
Plan 2	4,697	264	

<u>Comparison</u>	<u>Annual Incremental Benefits</u>	<u>Annual Incremental Costs</u>	<u>B/C</u>
Plan 1 vs. Existing	\$ 970	\$ 188	5.2
Plan 2 vs. Existing	1,033	218	4.7
Plan 2 vs. Plan 1	63	30	2.1

Both Plans are superior to the existing situation. However, when compared to each other, Plan 2 is superior to Plan 1.

## 5.7 DESIGN ELEMENTS

### 5.7.1 Design Vehicle

The Design Vehicle is the largest vehicle that frequently uses a facility. Projects with several types of facilities may have different design vehicles for each part. The physical and operating characteristics of the design vehicle are controlling parameters in highway design. Designs should accommodate the size and maneuverability of the design vehicle to allow it to operate without encroachment into adjacent travel or parking lanes. Such designs help reduce collisions and operational delays from lane encroachments.

#### 5.7.1.1 Minimum Design Vehicle for Various Routes

The geometric characteristics of the various design vehicles are found in Chapter 2 of AASHTO's, *A Policy on Geometric Design of Highways and Streets*, 2018. Below are the minimum required design vehicles for various highway categories for checking the horizontal alignment:

- For interstate highways, Designated Qualifying and Access Highways on the Designated Truck Access Highway Network, and their interchanges, the minimum design vehicle is the WB-67.
- For parkways, the design vehicle is the largest vehicle that will be regularly used on the highway, typically either an SU vehicle representing a maintenance vehicle or large school bus (S-BUS 40). (Note that Parkway are highways where commercial traffic is prohibited.)
- For most other non-interstate highways, the minimum design vehicle is the single unit truck (SU), which will also accommodate a large school bus (S-BUS 40). Some projects may require the larger city transit bus (CITY-BUS), articulated bus (A-BUS), WB-40, WB-62, WB-67, or larger design vehicle.

Based on the AutoTrack library for US Design Vehicles and the 2010 NCHRP 659 Ground Clearance Vehicles, below are the minimum required design vehicles for various highway categories for checking the vertical alignment:

- For arterial highways, including Interstates and other freeways; collectors; and railroad crossings on local roads and streets, the minimum design vehicle is the low boy trailer with a 5" minimum ground clearance.
- For local roads and streets (except at railroad grade crossings), the minimum design vehicle is the passenger car vehicle with a trailer since it governs the profile compared to a single unit truck (SU), large school bus (S-BUS 40), city transit bus (CITY-BUS), or articulated bus (A-BUS). Where low ground clearance vehicles are known to operate, the low boy trailer with a 5" minimum ground clearance may be considered.

#### 5.7.1.2 Encroachments

Vehicle encroachments occur when any portion of a vehicle extends beyond the vehicle's lane. With the exception of some low speed local streets and roads, designs that cause

frequent encroachments are undesirable as they may increase the likelihood of delays and collisions. However, designs that eliminate encroachments may also reduce safety since large turning radii allow faster turning speeds and wide turning paths create longer walking distances for pedestrians and may increase confusion for motorists confronted with large paved areas. In order to provide a balanced design, encroachments are generally acceptable for:

- Shoulders at intersections (Refer to [Chapter 3](#), Section 3.2.5.2 for a discussion of additional shoulder pavement thickness at intersections).
- Intersections along low-speed urban streets.
- Intersections along low-volume rural roads.
- Single left turns that require two receiving travel lanes.
- Double left and right turns that cannot accommodate side by side operation of the design vehicles. (Designs should accommodate a passenger car along side the design vehicle.)

#### 5.7.1.3 Oversized Vehicles

The selected design vehicle is often not the largest vehicle that may use the facility. Oversized vehicles require additional paved areas that are often not practical to construct due to the infrequency of these vehicles. We recognize that these oversized vehicles will occasionally be present and may encroach into other lanes and/or traverse shoulders and curbs. Designers should check proposed designs using the largest oversized vehicle anticipated to use the facility. This helps determine what changes would be needed to accommodate the oversized vehicle and helps decision makers determine whether or not such changes are practical.

Designers should contact their Regional Transportation Systems Operations Group to help determine an appropriate oversized vehicle. For many areas, the oversized vehicle is a modular home unit on a WB 20 trailer. The dimensions of the trailer load may be assumed to be a maximum of 16 ft (4.9 m) high including the trailer, 16 ft (4.9 m) wide, and 53 ft (17 m) to 80 ft (24.5 m) long. The wheel path for this vehicle can be easily checked using a WB 20 design vehicle. The vehicle overhangs should be checked to evaluate the location of trees, signals, poles, signs, shrubs, street appurtenances, etc.

When oversized vehicles encroach beyond the traveled way, the designer may need to consider:

- Traversable curb.
- Full depth shoulders.
- Wide shoulders.
- Stabilized areas behind curbing.
- Relocation of signals, poles, signs, trees, shrubs, street appurtenances, etc.
- Removable signs and street appurtenances.

As an alternative to a site-specific evaluation of oversized vehicles, the Region may coordinate with the Office of Safety and Security Services to develop alternative routing that bypasses a particular site.

## 5.7.2 Sight Distance

Sight distance is the length of road ahead visible to the driver. This distance should be long enough for the driver to see a situation and successfully react to it. There are a number of different types of sight distances important in highway design. Refer to Section 5.9.5 of this chapter for a discussion of intersection sight distance.

### 5.7.2.1 Stopping Sight Distance

Stopping sight distance is the distance necessary for a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. There are three types of stopping sight distance. These are: stopping sight distance for a crest vertical curve, stopping sight distance for a sag vertical curve (also called "headlight sight distance") and stopping sight distance for horizontal curves. Each of these types is equally important and only when all three conditions have been satisfied, can stopping sight distance requirements be considered satisfied. Stopping sight distance is one of the critical design elements and is discussed in [Chapter 2](#) and the "Standards for Non-Freeway Resurfacing, Restoration and Rehabilitation (3R) Projects". The NYSDOT "Vertical Highway Alignment Sight Distance Charts" in Appendix B of this chapter provide values for stopping sight distance and length of vertical curve for various algebraic differences in grade.

### 5.7.2.2 Passing Sight Distance

Passing sight distance is only a concern on two-lane, two-way roadways. On these highways, provision for passing is an important factor in maintaining the capacity of the highway. For a vehicle to pass a slower vehicle it has overtaken, it must occupy the lane regularly used by opposing traffic. To do so, the driver must be able to see far enough ahead to determine that the road is clear of opposing traffic and there is sufficient distance to complete the passing maneuver. Values for passing sight distance are found in Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. The NYSDOT "Vertical Highway Alignment Sight Distance Charts" in Appendix B of this chapter presents values for passing sight distance as a function of length of crest vertical curve and algebraic difference in grade.

### 5.7.2.3 Decision Sight Distance

Decision sight distance is the distance required for a driver to recognize a complex situation and safely react to it. Values for decision sight distance are substantially longer than for stopping sight distance. The increased sight distance is beneficial whenever the motorist encounters a condition which may increase the likelihood for error in information reception, decision-making, or control actions. The increased distance provides a greater margin for safety and is desirable where these kinds of errors are more likely, such as at interchanges and intersections, approaches to lane drops, and other locations where competing sources of information greatly complicate the tasks of driving. Further discussion and values for decision sight distance are found in Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

### 5.7.2.4 Horizontal Sight Distance

Concrete barriers and other similar items have grown in popularity in recent years. The effect of these barriers must be considered along with other visual obstructions when determining sight distance. A concrete barrier placed on the inside of a horizontal curve will restrict the sight distance around that curve. This is a common problem on curvilinear urban freeways with concrete median barrier. Refer to [Chapter 10](#), Section 10.2.2.5 of this manual for median barrier options.

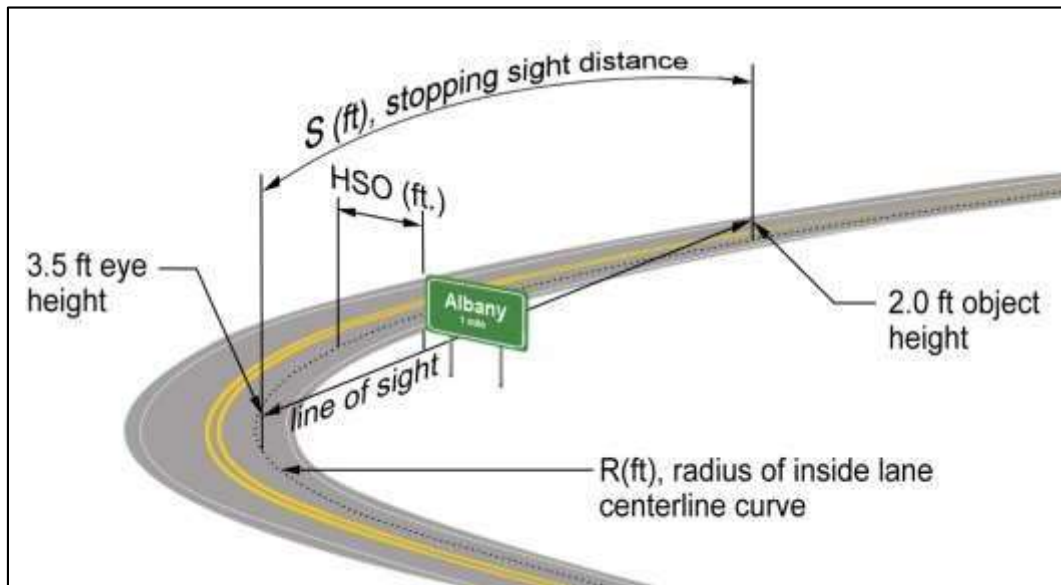
The following equation, a graphical method using CADD, or field measurements should be used to ensure sufficient stopping sight distance is provided along horizontal curves with obstructions, such as signs, concrete barrier, retaining walls, bridge abutments, etc. (refer to Exhibit 5-6). The equation is valid for curves with radii to the center of the inside lane that is equal or greater than the stopping sight distance. The eye height is 3.5 ft (1080 mm) and the object height is 2' (600 mm).

$$HSO = R (1 - \cos (28.65 S/R))$$

Where: HSO = Horizontal Sightline Offset  
 R = Curve Radius to Center of Inside Travel Lane  
 S = Stopping Sight Distance Measured Along the Travel Lane.

The above equation works for both metric and US Customary values. However, the units must be all in feet or meters.

**Exhibit 5-6 Horizontal Sight Distance**



### 5.7.3 Horizontal Curves

The horizontal alignment of a highway consists of a number of straight (tangent) sections connected by horizontal curves. These curves are sections of a circle or spiral. Curves should

be designed to minimize vehicles skidding off the traveled way (excessive vehicle yaw) or overturning (excessive vehicle roll). Refer to [Chapter 2](#) of this manual for curve radii, design speeds, and superelevation rates for various facilities.

### 5.7.3.1 Horizontal Curve Design to Minimize Vehicle Skidding Crashes

The point at which a vehicle begins to skid is based on a complex interaction of many variables, including:

- Traveled way superelevation, radius, grade, and coefficient of friction adjusted for weather, wear, and surface roughness.
- Vehicle mass, center of gravity, suspension, number of tires, velocity, antilock braking system, stability control system, steering angle, and acceleration/deceleration (i.e., accelerating or braking).
- Tire size, compound, tread design, wear, temperature, inflation, and contact patch.
- Motorist.

The following basic horizontal curve equation accounts for most of these variables (Ref. Equation 3-7 from AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018).

$$R = \frac{V^2}{15 (0.01e + f)}$$

The basic horizontal curve equation can be used to calculate:

- Radius, speed, or superelevation for new and reconstruction of all low-speed urban streets.
- The minimum radius for the selected design speed and maximum superelevation rate for new or reconstruction of rural highways and high-speed urban streets. Do **NOT** use the basic horizontal curve equation to determine the superelevation for intermediate curves (i.e., having radii greater than the minimum) on turning roadways, rural highways, and high-speed urban streets. Refer to [Chapter 2](#) of this manual for the applicable superelevation table for new and reconstruction of these facilities. Refer to [Chapter 7](#) for 1R, 2R and 3R projects.
- The recommended speed based on the radius and superelevation rate for all curves, as discussed in Section 5.2.5.1.B of this chapter. The recommended speed is shown in Exhibits 5-8 and 5-9.



**Exhibit 5-7 Side Friction Factor**

Speed (MPH)	f for NHS Facilities	f for non-NHS Facilities
20	0.27	0.30
25	0.23	0.29
30	0.20	0.28
35	0.18	0.27
40	0.16	0.26
45	0.15	0.25
50	0.14	0.24
55	0.13	0.23
60	0.12	0.22
65	0.11	0.21
70	0.10	0.20
75	0.09	0.19
80	0.08	0.18

Where:

R = Radius (ft). The horizontal curve radius to help prevent a vehicle from sliding out of its travel lane is based on a combination of the vehicle speed, side friction factor, and superelevation. The radius is measured from the inner edge of the traveled way on turning roadways and wide (4 or more lanes) undivided facilities. However, for two-lane undivided facilities, the radius may be measured to the centerline as the difference in radii is small. Similarly, for high-speed divided highways with separate horizontal control lines, the radius may be measured to the centerline of each direction of traffic..

V = Speed (mph). The approach design speed is used to determine the curve design speed.

f = Side Friction Factor from Exhibit 5-7. The side friction factor is the ratio of the lateral forces to the normal forces acting on a vehicle traveling around a curve. The friction factor is used to account for the complex interaction of the vehicle ( mass, center of gravity, suspension, number of tires, and velocity) and the traveled way (coefficient of friction adjusted for weather, wear, and surface roughness). Exhibit 5-7 shows the side friction factor for speeds of 20 mph through 85 mph. The NHS values were used to create Figures 3-3 and 3-4 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. For non-NHS facilities,  $f = 0.34 - 0.002 V$

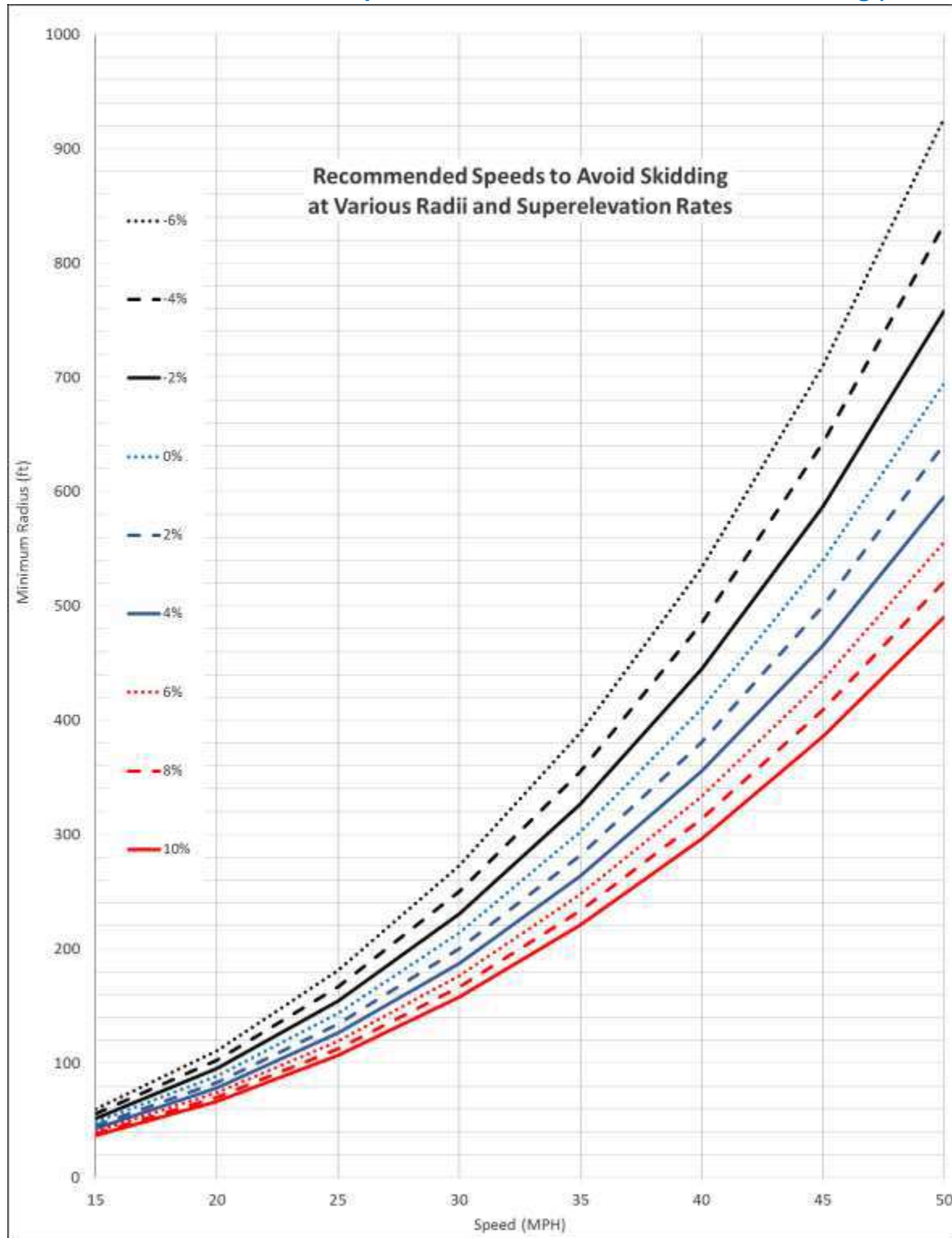
e = Superelevation in percent. The superelevation is the banking of the traveled way cross slope to counter the centrifugal forces of a vehicle traveling around a curve. The basic horizontal curve equation minimizes the use of superelevation, which minimizes the margin of safety since the cornering vehicle must use large amounts of side friction to avoid sliding off the curve. Refer to the superelevation distribution method 2 discussion in Chapter 3 of *AASHTO's A Policy on Geometric Design of Highways and Streets*, 2018.

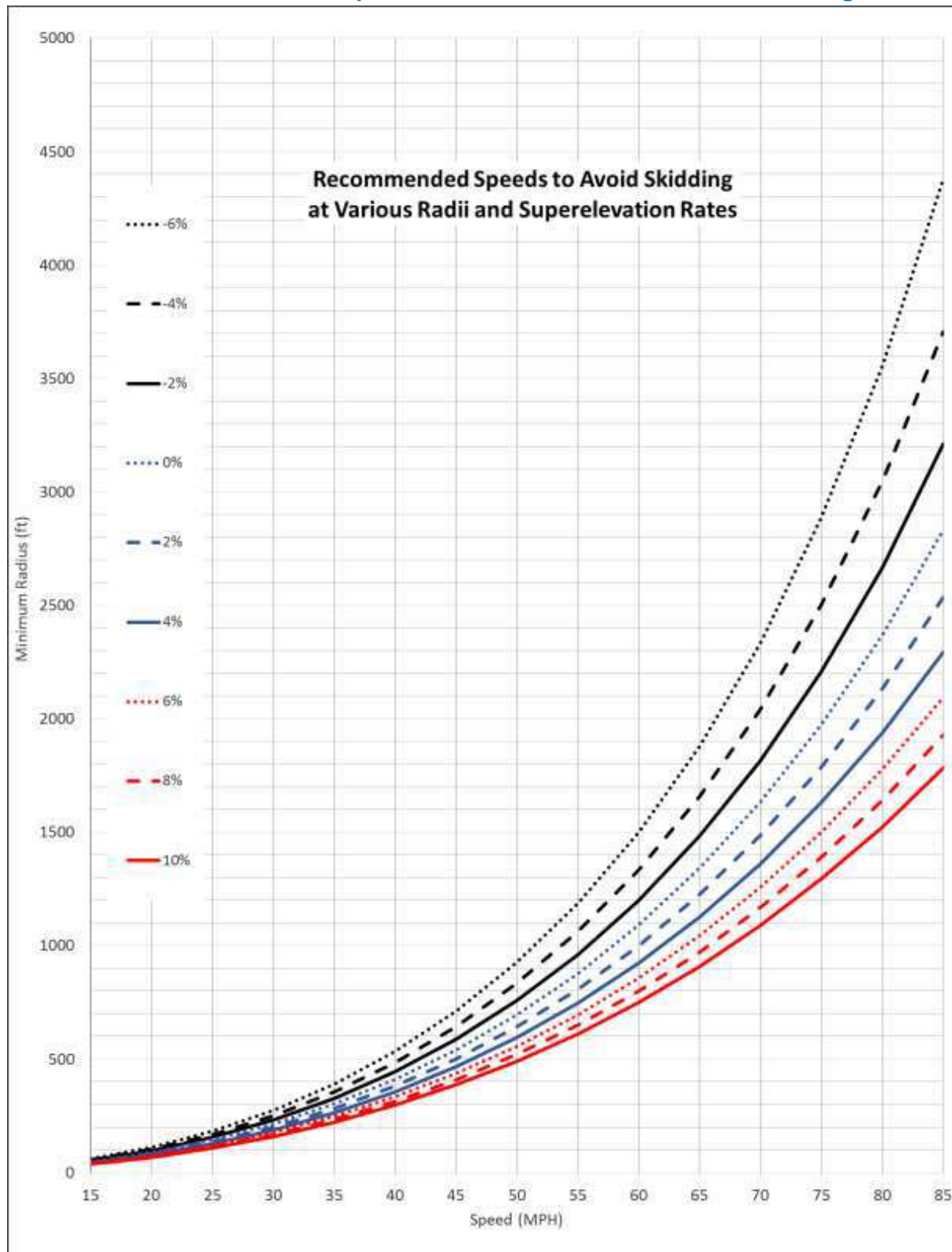
Exhibits 5-8 and 5-9 show the relationship between speed, superelevation and radius for non-NHS facilities using the linear side friction factor. It should be used as a lower threshold for nonstandard features on NHS facilities.

**Note:** The superelevation for intermediate curves on rural highways and high-speed urban streets is based on Superelevation Distribution Method 5, as discussed in Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Superelevation Distribution Method 5 uses a number of complex equations to place added superelevation on intermediate curves. As the radius decreases, the added superelevation allows the side friction demand to increase gradually, which increases the margin of safety over Superelevation Distribution Method 2. AASHTO's intent is to provide additional superelevation on curves with large radii to avoid violating driver expectancy. Motorists negotiating large radius curves are more likely to overdrive the curve and less likely to anticipate large cornering forces. The additional superelevation also allows the remaining available side friction to be used for changing roadway conditions, evasive maneuvers, braking, accelerating, etc.

The minimum radii and maximum percent of superelevation are found in [Chapter 2](#) of this manual for new construction, reconstruction, and bridge projects with over 400 ADT. For 1R projects and 2R/3R projects, refer to [Chapter 7](#) of this manual for allowable values. For bridge projects with 400 ADT or less, refer to [Chapter 4](#) of this manual. The maximum and minimum values should not be confused with desirable values. In new construction or reconstruction of high speed facilities, the largest radius possible is usually the most desirable solution.

When evaluating nonstandard horizontal curves, note that the side friction factor can be reduced by longitudinal forces from braking, accelerating, and the increased tractive forces needed to maintain speed on steep inclines. Where these actions are likely, additional superelevation and other measures should be considered.

**Exhibit 5-8 Recommended Speed on Horizontal Curves to Avoid Skidding (Low Speed)**

**Exhibit 5-9 Recommended Speed on Horizontal Curves to Avoid Skidding**

### 5.7.3.2 Horizontal Curve Design to Avoid Vehicle Rollover Crashes

The typical passenger car will skid long before it rolls over on the pavement, particularly in wet weather. Trucks, vans, and sport utility vehicles have much higher centers of gravity and may rollover before skidding, particularly in dry weather and at lower speeds. Vehicle rollover is generally not a limiting factor that influences horizontal curves meeting current standards. Vehicle rollover should be considered for nonstandard curves, particularly when they are likely to violate driver expectancy (such as a nonconforming compound curve), which is discussed in Section 5.8.3 of this chapter.

The point at which a vehicle begins to rollover is based on a complex interaction of many variables, including those listed in Section 5.7.3.1 of this chapter. The horizontal curve equation to determine the point of impending rollover is:

$$R = \frac{V^2}{15 (0.01e + a)}$$

Where:

R = Radius (ft). The horizontal curve radius to help prevent a vehicle from rolling over is based on a combination of the vehicle speed, superelevation, and the maximum allowable lateral acceleration for given vehicle. The radius is measured from the inner edge of the traveled way on turning roadways and wide (4 or more lanes) undivided facilities. However, for two-lane undivided facilities, the radius may be measured to the centerline as the difference in radii is small. Similarly, for high-speed divided highways with separate horizontal control lines, the radius may be measured to the centerline of each direction of traffic.

V = Speed (mph). The approach design speed is used to determine the curve design speed.

e = Superelevation or banking of the traveled way cross slope (%).

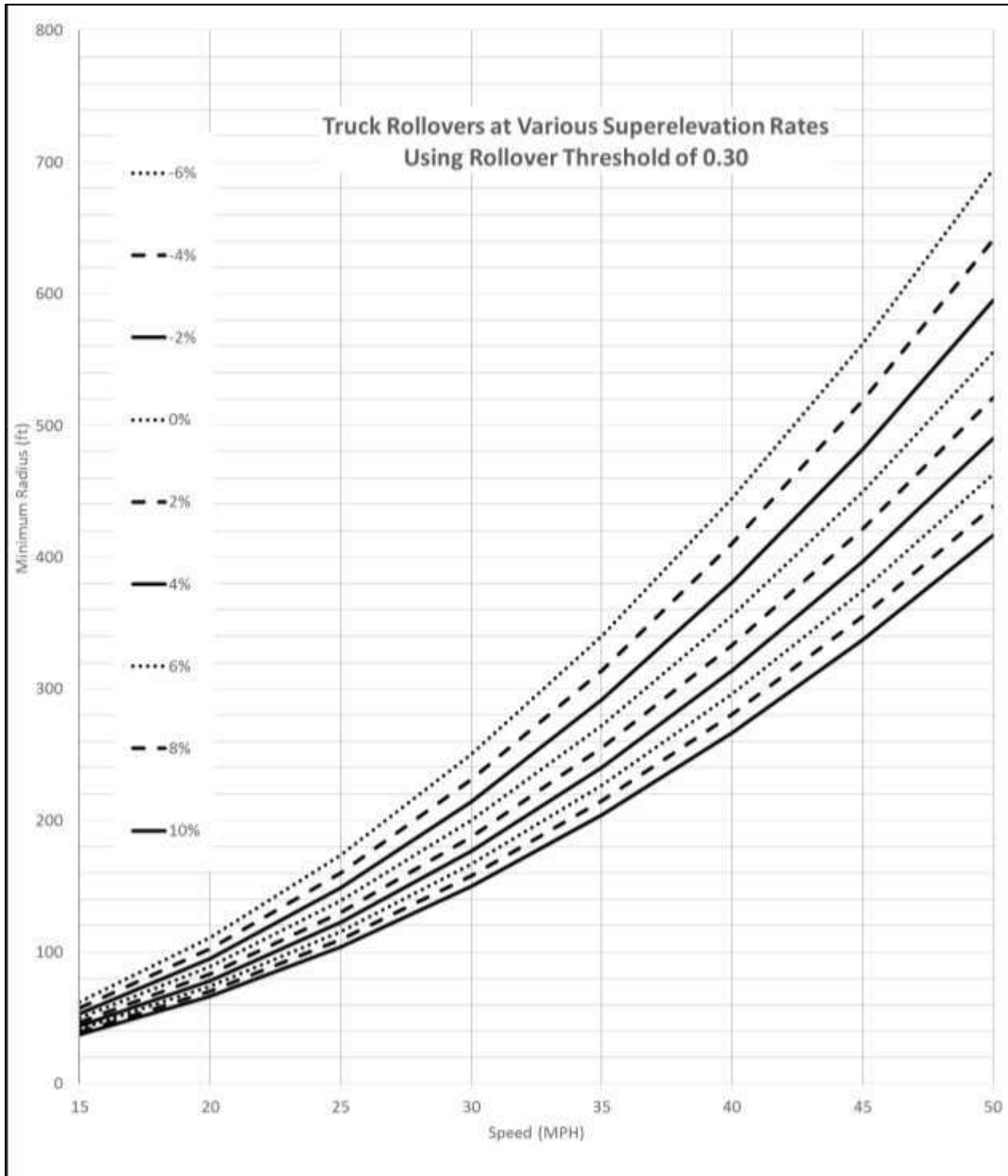
a = The approximate rollover threshold.

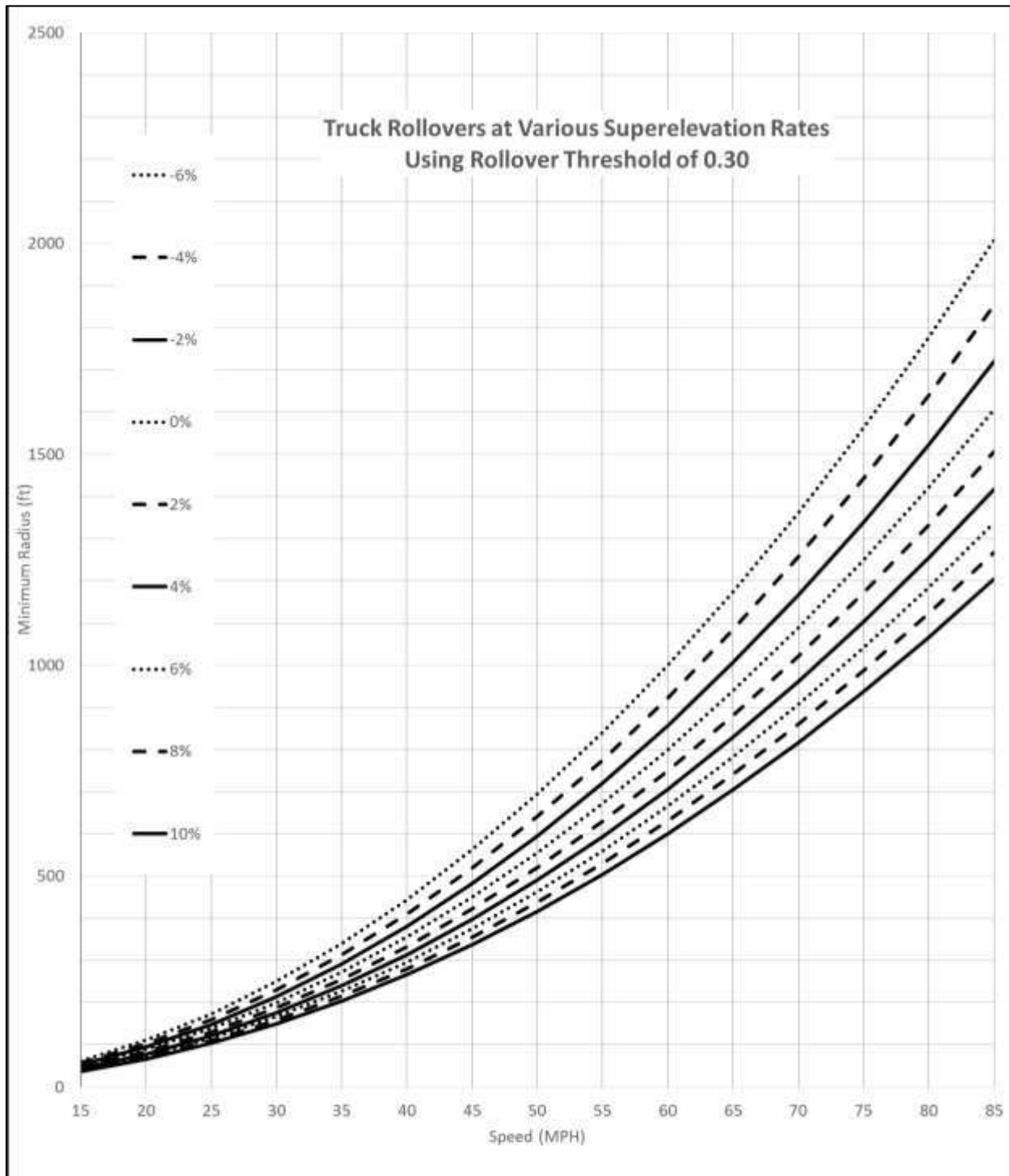
Studies referenced in NCHRP 774 *Superelevation Criteria for Sharp Horizontal Curves on Steep Grades*, 2014, found that the worst case rollover thresholds for trucks are approximately 0.35. A value of 0.30 is used in the tables to approximate 85% of the rollover threshold.

Solutions to an existing truck rollover problem may include:

- Provide additional signing with flashing lights.
- Install a truck rollover warning system (flashing lights are activated by truck approach speeds).
- Increase the superelevation up to a maximum of 8.0%.
- Increase the lane width. A wider lane width allows maneuvering area for large vehicles to steer to the outside of the curve for a brief moment to induce a righting force and to reduce speed.
- Reconstruct the horizontal alignment to current standards.

**Exhibit 5-10 Recommended Speed on Horizontal Curves to Avoid Truck Rollovers (Low Speed)**



**Exhibit 5-11 Recommended Speed on Horizontal Curves to Avoid Truck Rollovers**



### 5.7.3.3 Superelevation Transitions

The purpose of transitions at the ends of horizontal curves is to change the cross slope from normal crown to full superelevation and back. The length of these transitions should be chosen to provide a smooth-riding and pleasant-appearing transition. Superelevation transitions can be achieved over a tangent-spiral or tangent-circular curve combination. In both cases, the length of the transition is found in Exhibit 5-15 in this section. Exhibits 5-13 and 5-14 indicate the methods of attaining superelevation.

Refer to [Chapter 3](#), Section 3.2.5.1 of this manual for the shoulder cross slope along superelevated sections.

#### A. Runoff

In a transition, the runoff is the distance used to change the section from the point where adverse crown is removed (the high side is level) to the point where full superelevation is achieved. The runoff ( $L_r$ ) is determined from the lane width ( $w$ ) in feet, number of lanes ( $n_l$ ), percent superelevation ( $e_d$ ), an adjustment factor for the number of lanes to be rotated ( $b_w$ ), and the maximum relative gradient ( $\Delta$ ) from Exhibit 5-12, in percent.

Superelevation Runoff Equation: 
$$L_r = \frac{w e_d n_l b_w}{\Delta}$$

**Exhibit 5-12 Maximum Relative Gradient**

Design Speed (mph) $V_d$	Maximum Relative Gradient (%) $\Delta$
15	0.80
20	0.80
25	0.74
30	0.67
35	0.65
40	0.57
45	0.54
50	0.50
55	0.50
60	0.50
65	0.50
70	0.50
75	0.50
80	0.50



Exhibit 5-13 Method of Attaining Superelevation

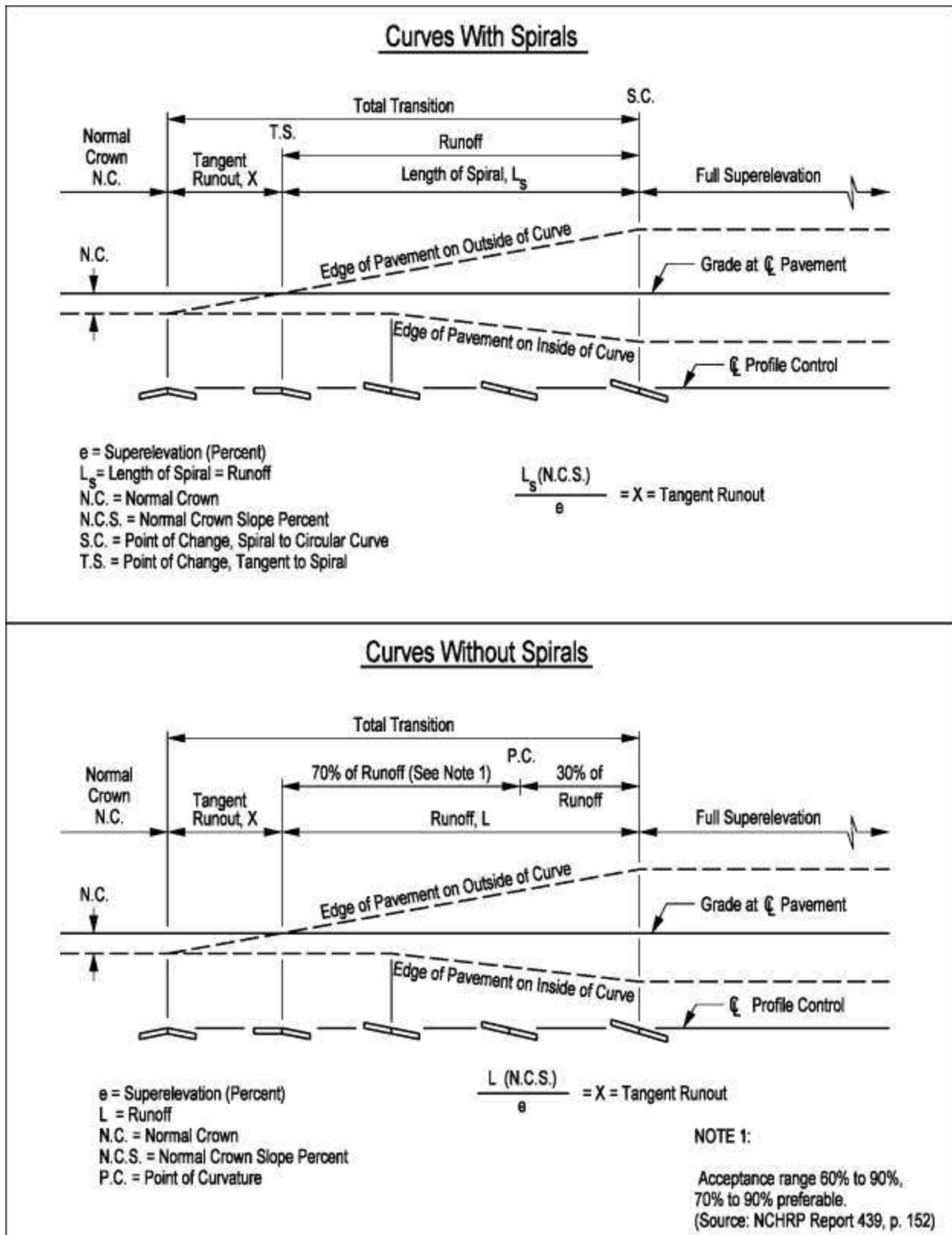
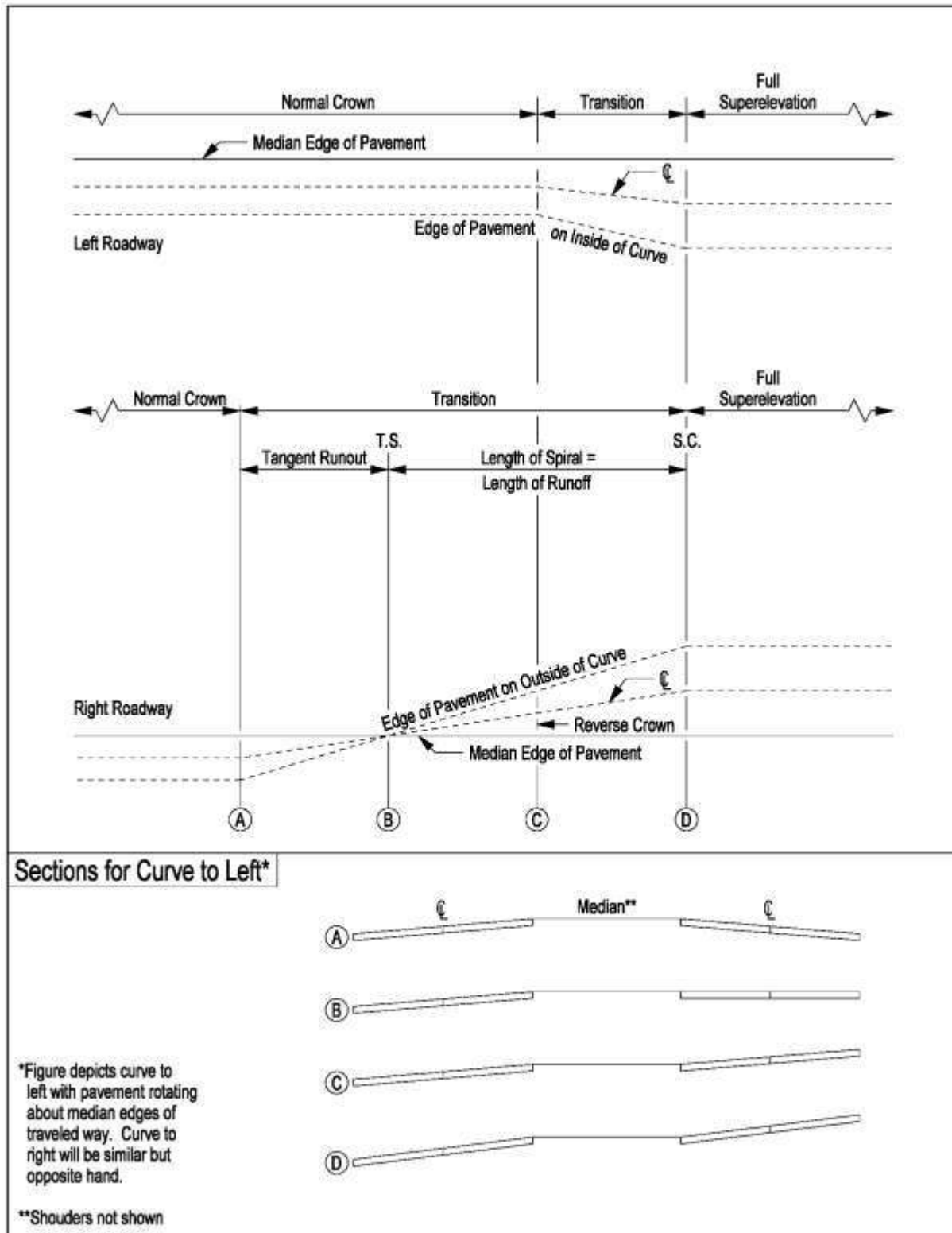


Exhibit 5-14 Method of Attaining Superelevation – Four Lane Divided Highway\*



The length of runoff for 1 lane rotated and 2 lanes rotated should be determined from the superelevation runoff equation or by Exhibit 5-15, which assumes a 12' (3.6 m) wide lane. For other situations, the runoff equals the length of 1 lane rotated multiplied by the factor ( $n_1 b_w$ ) provided below:

Number of Lanes Rotated	Length to Increase Relative to 1 Lane Rotated ( $n_1 b_w$ )
1	1
1.5	1.25
2	1.5
2.5	1.75
3	2
3.5	2.25

A spiral is a curve of constantly changing radius. The radius at each end matches the alignment into which it is going. A spiral curve provides a smooth, natural path for a vehicle entering or leaving the circular curve. The spiral curve is used to achieve only the runoff portion of the banking transition. The runout portion is placed on the tangent.

As a general rule, spirals are to be used for new or reconstructed curves. Introducing spiral transitions in curves that do not have them will shift the center line of the circular curve toward the center of the curve radius.

#### B. Runout

Runout is the change from a normal crown section to a section with no adverse cross slope (i.e.,  $e = 0$  on the high side of the traveled way). To effect a smooth edge of pavement profile, the rate of removal should equal the relative gradient used for the superelevation runoff. For sections with -2% cross slope, the runout length can be determined using the 2.0% superelevation row in Exhibit 5-15, adjusted as needed for the number of lanes rotated. For 1.5% or other cross slopes, the runout length should be determined using the runoff equation provided in Section 5. 7.3.3.A using a percent superelevation ( $e_d$ ) equal to the cross slope.

To avoid ponding storm water on the traveled way, a minimum grade of 0.5% should be maintained where the transition has travel lanes superelevated less than the normal cross slope.

**Exhibit 5-15 Superelevation Runoff  $L_r$  (ft) for Horizontal Curves**

$V_d$	Lanes <sup>1</sup>	e (%)	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0
15 mph	1	$L_r$ (ft)	23	30	38	45	53	60	68	75	83	90	98	105	113	120
	2	$L_r$ (ft)	34	45	56	68	79	90	101	113	124	135	146	158	169	180
20 mph	1	$L_r$ (ft)	23	30	38	45	53	60	68	75	83	90	98	105	113	120
	2	$L_r$ (ft)	34	45	56	68	79	90	101	113	124	135	146	158	169	180
25 mph	1	$L_r$ (ft)	24	33	41	49	57	65	73	82	90	98	106	114	122	131
	2	$L_r$ (ft)	37	49	61	73	86	98	110	122	135	147	159	171	184	196
30 mph	1	$L_r$ (ft)	27	36	45	54	63	72	81	90	99	108	117	126	135	144
	2	$L_r$ (ft)	40	54	67	81	94	108	121	135	148	162	175	189	202	216
35 mph	1	$L_r$ (ft)	28	37	46	55	64	73	83	92	101	110	119	129	138	147
	2	$L_r$ (ft)	41	55	69	83	96	110	124	138	152	165	179	193	207	220
40 mph	1	$L_r$ (ft)	32	42	53	63	74	84	95	105	116	126	137	147	158	168
	2	$L_r$ (ft)	47	63	79	95	110	126	142	158	173	189	205	221	236	252
45 mph	1	$L_r$ (ft)	34	45	56	67	78	90	101	112	123	134	146	157	168	179
	2	$L_r$ (ft)	50	67	84	101	118	134	151	168	185	202	218	235	252	269
50 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
55 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
60 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
65 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
70 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
75 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
80 mph	1	$L_r$ (ft)	36	48	60	72	84	96	108	120	132	144	156	168	180	192
	2	$L_r$ (ft)	54	72	90	108	126	144	162	180	198	216	234	252	270	288
<b>Note:</b>																
1. Lanes are the number of lanes rotated. Note that 1 lane rotated is typical for a 2-lane highway, 2 lanes rotated is typical for a 4-lane highway.																

#### 5.7.3.4 Widening Along Horizontal Curves

Travel lane widening along horizontal curves compensates for vehicle off-tracking, steering difficulty, and lane infringement. The need for travel lane widening is common along relatively sharp horizontal alignments. This need is exacerbated by narrow lane widths, narrow shoulder widths, or the lack of spiral transitions. It applies to both one-way and two-way facilities. It does not apply to turning roadways at intersections or interchanges, which are to be designed using Table 2-9 in [Chapter 2](#) of this manual.

##### A. Benefits of Widening Along Sharp Horizontal Curves

Table 7 in FHWA's *Safety Effectiveness of Highway Design Features, Volume II: Alignment*, November, 1992, shows that the crash rate can be reduced by 5% to 21% by widening the traveled way along horizontal curves. Widening or providing paved shoulders along a horizontal curve can also result in substantial crash reductions.

##### B. Design of Widening Along Sharp Horizontal Curves

Refer to Table 3-24a in AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, for the recommended pavement widening along horizontal curves for a WB-62 design vehicle. Adjustments should be made for different design vehicles using Table 3-25.

Although paved shoulders provide some compensation when travel lanes are not widened along sharp horizontal curves, the highway should be designed so that vehicles only use the shoulder in emergency situations. When the right of way is severely constrained and paved shoulders are provided, a portion of the paved shoulder width may be subtracted from the above values since drivers can use part of the paved shoulder to increase the offset between passing vehicles. However, if there is frequent truck traffic (>10%), bicycle traffic, or a history of side-swipe, run-off-the-road, head-on, fixed-object, or rollover crashes, the pavement widening values should be used.

When widening the traveled way, the additional paved width should be rounded to the nearest foot (tenth of a meter) and added:

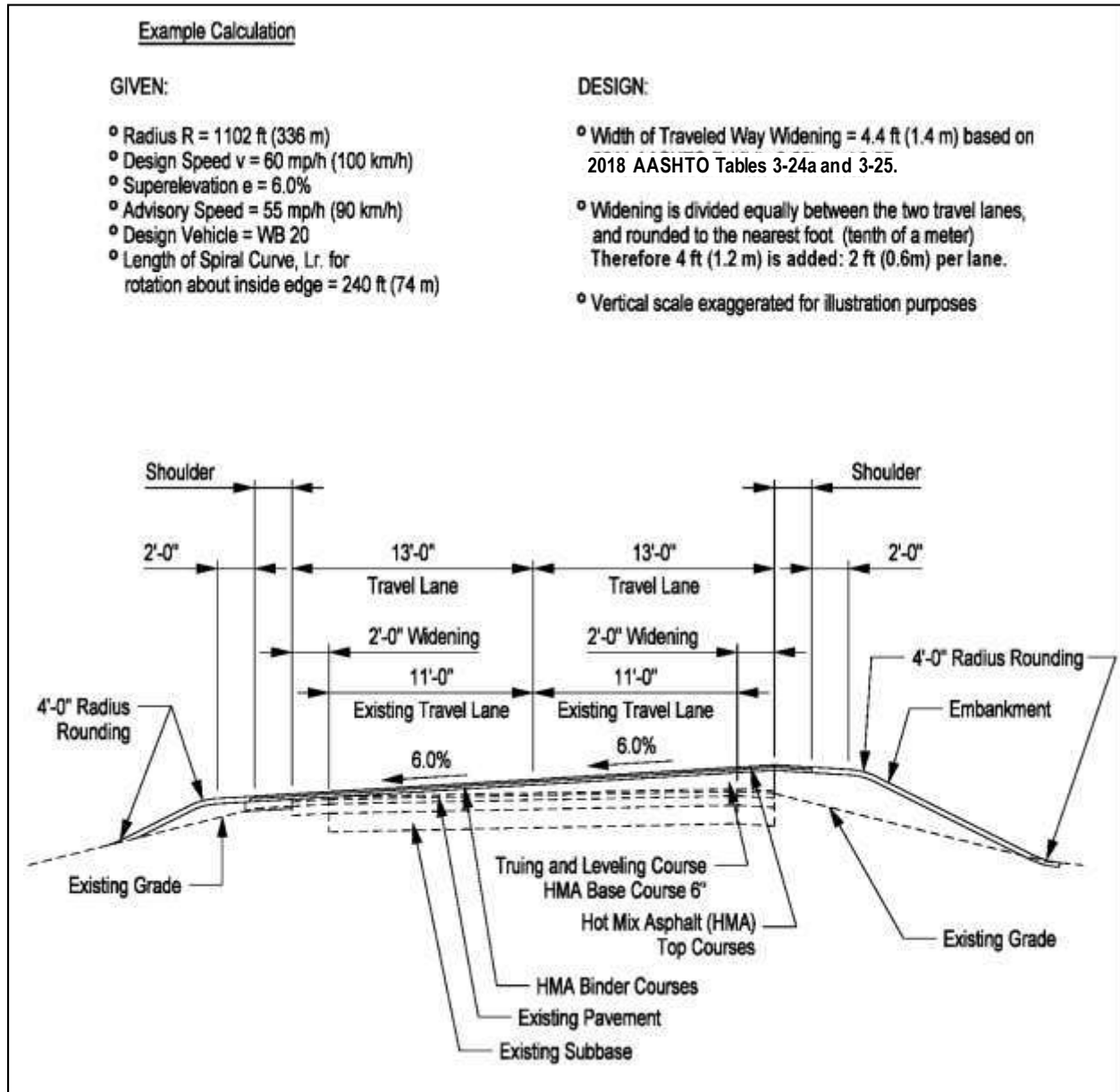
- Equally to both sides of the curve along spiraled curves, as shown in Exhibit 5-16, or
- To the inside edge of the curve for curves without spiral transitions, as shown in Exhibit 5-17.

The pavement structure of the traveled way widening should be designed to meet the rigors of the additional vehicular traffic. Since the traveled way widening is often directly over the shoulder, the existing shoulder may require removal and replacement with the appropriate course(s) where the shoulder is severely deteriorated, unpaved, or inadequate to handle the projected traffic.

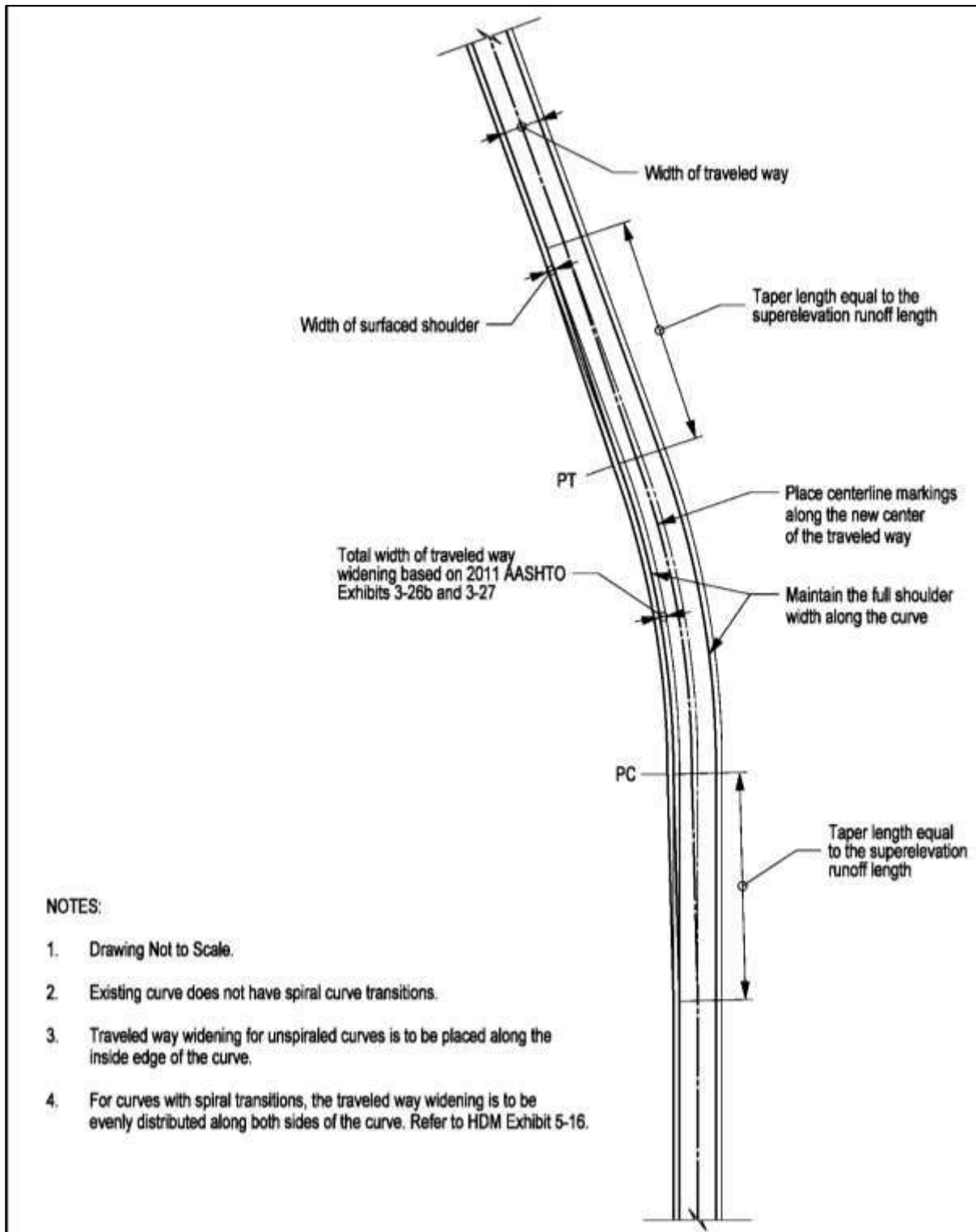
As shown in Exhibits 5-16 and 5-17, the centerline markings should be placed along the centerline of the final, surfaced roadway. The edge striping should be located so that the normal shoulder width, from the tangent or un-widened curved sections, are maintained

along the curve to permit use of the shoulder by bicyclists, pedestrians, and stopped vehicles.

**Exhibit 5-16 Cross Section View of Travel Lane Widening Along Spiral Curves**



**Exhibit 5-17 Plan View of Travel Lane Widening Along Curves Without Spiral Transitions**



### 5.7.3.5 Combinations of Curves

Avoid combinations of circular curves occurring together, whenever possible. There are a number of special types of these combinations.

#### A. Compound Curves

A compound curve is two or more curves of different radii but curving in the same direction and connected together. Larger radius curves followed by lesser radius curves are of special concern because of inconsistencies with driver expectation.

When this cannot be avoided, the ratio between the successive curve radii should be limited when the tighter radius curve  $\leq 2$  times the minimum horizontal curve radius as follows:

Ramps and Mainline Curves: 1:2 Maximum  
Mainline Curves: 1:1.5 Desirable

#### B. Broken-Back Curve

A broken-back curve is two curves, turning in the same direction, with a short tangent between them. On new construction or reconstruction, a minimum tangent of 1,500 ft (450 m) should be provided between curves turning in the same direction.

#### C. Reverse Curves

A reverse curve is two curves, turning in opposite directions, and connected together. A tangent section between reverse curves that is of sufficient length to provide for full runoff and runout for both curves is desirable. If sufficient distance (i.e., more than 325 ft [100 m]) is not available to permit the tangent runout lengths to return to a normal crown section, a large area can be at the same plane with the edges of pavement and centerline at the same elevation. This condition results in poor transverse drainage. To prevent drainage problems, the superelevation runoff lengths should be increased until they abut, thus providing one instantaneous level section. The pavement is continuously rotated from full superelevation in one direction to full superelevation in the other. If the minimum superelevation runoff lengths cannot be obtained for each curve, realignment should be considered.

## 5.7.4 Vertical Alignment

### 5.7.4.1 Grades

Grades affect the operating characteristics of vehicles. Braking distances increase on downgrades and decrease on uphill grades. It is difficult for heavy vehicles to maintain their speed on steep- uphill grades. Consideration should be given to adjusting the required stopping sight distance to account for the effects of grade. Chapter 3 of AASHTO's *A Policy*



*on Geometric Design of Highways and Streets*, 2018, contains a detailed discussion of the effects of grade.

The maximum allowable grade on a roadway is generally determined by its functional classification, the design speed, and the terrain. See [Chapter 2](#) of this manual for the appropriate values and a discussion of terrain and design speed.

The minimum grade will generally be controlled by the design of the drainage system. In cut sections, it is desirable to have a minimum grade of 0.5% to avoid constructing special ditches. While flatter grades may be acceptable in some situations, curbed roadways and superelevation transition sections with less than 1% of cross slope should have a minimum grade of 0.5%. In fill sections, a level profile may provide adequate drainage.

When evaluating the vertical alignment, the placement of a sag vertical curve on a structure should be avoided whenever possible due to fabrication problems as well as drainage problems on curbed structures.

#### 5.7.4.2 Vertical Curves

Vertical curves provide for a gradual change in grade between the approach tangents. Vertical curves should be designed to provide sufficient sight distance, comfortable operation, efficient drainage, and a pleasant appearance. Long vertical curves generally have a more pleasing appearance than short vertical curves. When faced with a choice, designers should use shorter sag vertical curves in favor of providing longer crest vertical curves.

##### A. Crest Vertical Curves

The provision of adequate sight distance for the design speed is the primary factor in the safe operation of crest vertical curves. The minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, additional sight distance should be provided. AASHTO recommends providing the desirable stopping sight distance in these instances.

For appearance and comfort, even small changes in grade generally warrant a vertical curve. The minimum length of crest vertical curves should be the length needed to provide the minimum stopping sight distance.

Where the change in grade is very slight, a vertical curve may not be practical. Generally, a vertical curve is not needed if the algebraic change in percent grade (G) is equal to or less than 1.6 minus 1/50<sup>th</sup> of the design speed (V<sub>d</sub>) in miles per hour.

$$G \leq 1.6 - 0.02 \times V_d$$

## B. Sag Vertical Curves

Four criteria are used to establish the minimum length of sag vertical curves. They are sight distance, drainage, riding comfort, and appearance. Minimum lengths for drainage and sight distance under vertical obstructions are needed; minimum lengths for riding comfort and appearance are desirable.

### Sight Distance

When a vehicle traverses a sag vertical curve at night, high-beams, tail lights of other vehicles, or street lighting provide sight distance. However, where sight distance is reduced due to overhead structures, such as an overpass or sign structure, the required stopping sight distance is needed. Exceptions require a non-standard feature justification in accordance with Section 2.8 of this manual.

### Drainage

In sag vertical curves, the surface drainage of curbed pavements requires special attention. Flat gradients may result in ponding. The design of the drainage system may control the minimum vertical curve length on curbed roadways. Generally, providing a minimum grade of 0.30% within 50 ft. (15 m) of the level point ensures adequate roadway surface drainage. Refer to [Chapter 8](#), Section 8.7.4.4 of this manual.

### Riding Comfort

Riding comfort is the effect of the change in vertical direction. The effects of riding comfort are greater on sag vertical curves than on crest vertical curves. The gravitational and centrifugal forces are combining rather than opposing. There is a detailed discussion of riding comfort in Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018

### Appearance

The need for a minimum length of vertical curve may be based on appearance. For this purpose, AASHTO recommends a minimum length of 3 times the design speed (0.6 times the design speed for metric units)

## C. Intersections on Vertical Curves

Vertical and horizontal sight distances are crucial elements in the design of intersections. At all design speeds, the sight distance needed to perform entering and crossing maneuvers is considerably longer than the minimum stopping sight distance. For further discussion, see Section 5.9, Intersections at Grade.

### 5.7.5 Climbing Lanes

A climbing lane is an additional lane provided to permit the passing of slow -moving traffic in the uphill direction. Its purpose is to improve the operational and safety characteristics of the roadway. It is desirable to provide a climbing lane when the grade, traffic volume, and the heavy vehicle volume combine to significantly degrade traffic operations. Heavy vehicles are those with a mass to power ratio of 200 lb/hp (120 kg/kW) or greater.

On two-way, two lane roadways, meeting the following three conditions would justify a climbing lane. However, other conditions may arise on low-volume highways where sufficient passing opportunities are not available where it might be advantageous to provide a climbing lane even though the following warrants are not met.

1. An upgrade traffic flow rate in excess of 200 vph.
2. An upgrade truck flow rate in excess of 20 vph.
3. One of the following conditions exists:
  - a. A 10 mph (15 km/h) or greater speed reduction is expected for a typical heavy truck based on Table 3-26a of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.
  - b. The level of service on the grade is E or F.
  - c. A reduction of two or more levels of service between the approach and the grade.

On multilane highways and freeways, a climbing lane is justified when both of the following conditions are met:

1. A 10 mph (15 km/h) or greater speed reduction is expected for a typical heavy truck based on Table 3-26a of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.
2. Either of the following conditions is met:
  - a. There is a drop of more than one level of service between the desired design level of service and the level of service on the grade.
  - b. The level of service on the grade is E or F.

The point of need for the climbing lane is the spot where the truck operating speed is reduced 10 mph (15 km/h). The climbing lane should be developed through an abrupt 150 ft (45 m) taper starting 250 ft (75 m) before the point of need.

Ideally, the climbing lane should be extended beyond the crest to a point where the truck operating speed is within 10 mph (15 km/h) of the highway operating speed. Due to the poor acceleration characteristics of heavy trucks, it may be impractical to obtain the desired length. In these cases, the climbing lane should be extended as far as practicable. For the minimum passing sight distance refer to Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. For the appropriate taper lengths, refer to Table 6H-4 of the [National MUTCD](#).

### 5.7.6 Emergency Escape Ramps

Long, descending grades distinctly increase the potential for heavy vehicles to experience loss of braking ability. For grades where this is an identified problem, a properly designed emergency escape ramp at an appropriate location can safely slow and stop out-of-control vehicles away from the main traffic stream. Sand piles, arrester beds, dragnets, and ascending-grade gravity ramps, alone or in combination may be used.

Refer to Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, and [Chapter 10](#) of this manual for a general discussion of emergency escape ramps. See Chapters [10](#) and [16](#) of this manual and Chapter 8 of AASHTO's *Roadside Design Guide* for specific information on arresting devices and attenuation systems.

### 5.7.7 Travel Lanes and Shoulders

Refer to [Chapter 2](#) of this manual for the minimum and desirable travel lane and shoulder widths. Refer to [Chapter 3](#) of this manual and the [Comprehensive Pavement Design Manual](#) for information on pavement sections.

### 5.7.8 Lane Drops

Lane drops are used whenever it becomes necessary to reduce the number of through lanes. Proper lane balance should be maintained to reduce bottlenecks and reoccurring congestion. A capacity analysis should be performed to evaluate the consequences of any lane reduction.

On freeways, the lane reduction should be effected between interchanges or at a two-lane exit ramp. To allow for adequate signing, the lane reduction should be located at least 2,000 ft. (600 m) to 3,000 ft. (900 m) from the previous interchange and beyond any acceleration lane. The reduction should not be made so far downstream that motorists become accustomed to the number of lanes and are surprised by the lane reduction. Visibility of the lane reduction is an important consideration. Desirable locations are on tangents, on approaches to crest vertical curves, and on the uphill side of sag vertical curves. Lane drops on moderate to sharp horizontal curves should be avoided.

AASHTO suggests that the lane reduction be on the right side of the roadway. Speeds are generally lower in the right-hand lane and the merging maneuver is more common than a merge from the left. Following exit ramps, there is usually less traffic in the right-hand lane.

The lane drop shall be tapered. The minimum taper length should be in accordance with the merge taper lengths in Table 6H-4 of the [National MUTCD](#).

A three- or four-lane highway transitioning to a two-lane, two-way roadway produces another lane drop situation. The lane shift can be centered or placed on either side. The appropriate signing and pavement markings for these situations are shown in the National MUTCD.

### 5.7.9 Medians

Medians are desirable for streets with four or more traffic lanes. The primary functions of medians are to provide the following:

- Pedestrian safety when used and functionally designed as a refuge area
- Storage space for left-turning vehicles
- Separation of opposing traffic streams
- Access control to/from minor access drives and intersection
- Traffic calming

A median is defined as the portion of a divided highway separating the traveled way for traffic traveling in opposing directions. The median width is expressed as the dimension between the through-lane edges and includes the left shoulders, if any. Median width is a design consideration only for interstates, other freeways, and multilane divided rural arterials. An arterial is not normally considered to be divided unless two travel lanes are provided in each direction of travel and the median has a width of 4 ft. or more and contains a barrier, turf, raised sections, or lowered sections to preclude its use by motorists, except in emergencies or where the median is specifically designed to allow for left turns.

For additional information on medians, refer to [Chapter 3](#), Section 3.2.8 of this manual. Refer to this chapter, Section 5.9.8.2C for information on Two-Way Left-Turn Lanes. Refer to [Chapter 18](#) for the design of medians for pedestrian refuge.

**Exhibit 5-18 Minimum Median Widths**

Classification	Character	Terrain	Width
<b>Interstates &amp; Other Freeways</b>	Rural	Level or rolling	36 ft. min. 50-100 ft. desirable
	Rural	Mountainous	10 ft. min
	Urban	All	10 ft. min
<b>Arterials</b> (multilane, divided, rural arterials only)	Rural	All	Without left turn lanes - 4 ft. min. With left turn lanes - 12 ft. min. (10 ft. left turn lane with 2 ft. median separation)

### 5.7.10 Median - Emergency Crossovers

Median crossovers are needed to facilitate maintenance and emergency operations on controlled-access facilities. Maintenance crossovers may be required at one or both ends of an interchange. Crossovers may be provided on rural freeways when the interchange spacing exceeds 5 miles (8 km). Generally emergency crossovers are placed every 3 miles to 4 miles (5 km to 6.5 km) between interchanges. The placement of the crossovers should be coordinated with the Regional Highway Maintenance Engineer. The appropriate police and emergency services should be contacted for their input.

Maintenance and emergency crossovers should not be located within 1,500 ft (450 m) of the end of a ramp. Crossovers should be situated at locations where decision sight distance is available. If possible, they should not be located on curves requiring superelevation.

To minimize the effect on an out-of-control vehicle, crossovers should be constructed with seeded side slopes of 1:10 or flatter. A rounding with a 50 ft (15 m) radius should be provided at the crossover's toe of slope and at the intersection of the mainline fill and the crossover fill. If drainage is carried through the median in a pipe with a diameter greater than 1 ft (300 mm) at a location accessible to an errant vehicle, a slanted grate should be used over the beveled end of the pipe. The 50 ft (15 m) rounding and the slanted grate may be eliminated when guide railing on the mainline adequately shields the motorist from the hazard.

The minimum recommended crossover width is 25 ft (7.5 m). On narrow medians, a greater width of pavement may be necessary to safely accommodate turning vehicles. Crossovers should not be used in restricted-width medians. The median must be wide enough to store a typical maintenance vehicle. The surface and shoulders should be designed to support the appropriate maintenance equipment.

A parallel-type deceleration lane should be provided. See Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Its length should be determined from this AASHTO policy. It should be designed to accommodate the appropriate maintenance vehicle and surfaced with the same type of material and have the same cross slope as the mainline shoulder. A design speed of 60 mph (100 km/h) should be used and a 4 ft (1.2 m) shoulder provided. The design speed may be decreased on low-speed facilities. An acceleration lane is normally not provided. However, in special circumstances, an acceleration lane may need to be evaluated, depending on traffic volume, speed, crash history, etc.

When the crossover is constructed in an area with a median barrier, the median barrier should be designed to limit the hazard. Opposing end sections should be shielded from oncoming traffic. It may be necessary to design the barrier to guide vehicles away from the median opening or to use impact attenuators. See [Chapter 10](#), Section 10.2.5 of this manual for suggested flare rates.

### 5.7.11 Roadway Clear Zone

A clear unobstructed roadside is highly desirable. The term "clear zone" is used to designate the width that the Department has committed to maintain as an unobstructed, traversable area provided beyond the edge of the traveled way for the recovery of errant vehicles.

The desirable width of the clear zone is influenced by the traffic volumes, speed, horizontal curvature, and embankment slopes. Although AASHTO has established desirable design values for clear zone widths, actual values may vary for different projects or project segments. Project-specific values, determined by sound engineering judgment during design, shall be documented in the Design Approval Document.

Most Department projects require the establishment of design clear zone widths. See [Chapter 10](#) of this manual for specific requirements and guidance.

### 5.7.12 Vertical and Horizontal Clearances

Vertical and horizontal clearances are important elements in the design of a highway. Clearances to be considered are:

- Vertical over and horizontal along a roadway.
- Vertical over and horizontal beside a railroad.
- Vertical over and horizontal beside a waterway.
- Vertical and horizontal between a roadway and an airway.

#### 5.7.12.1 Roadway

The policy for vertical clearance over a roadway is stated in Section 2 of the Department's [Bridge Manual](#). Vertical clearance is a critical design element and is discussed in [Chapter 2](#) of this manual. Vertical clearance is the minimum vertical clear distance to an obstruction over any part of a highway's pavement or shoulders.

Horizontal clearance is a segment of the road section lying adjacent to the traveled way, identified as an operational offset in urban areas and for rural areas identified as a portion of the “clear zone” (defined in [Chapter 10](#) of this manual as an area for recovery of errant vehicles). It does not replace the need to select a clear zone (in accordance with Chapter 10 of this manual and AASHTO's *Roadside Design Guide*) that will generally be substantially wider than the horizontal clearance criteria in this chapter. A more detailed description of what features are allowed within these two categories follows.

##### A. Interstates, Other Freeways, Expressways, Rural Arterials, Rural Collectors, and Local Rural Roads

Horizontal clearance serves as an extension of the shoulder and provides allowance for recovery of errant vehicles, disabled vehicles, parking, etc. Curbs, traversable slopes, breakaway supports, etc., are permitted within the horizontal clearance. Fixed objects, nontraversable slopes, etc., are not permitted. The width is measured from the edge of traveled way. It includes shoulders or auxiliary lanes (e.g., speed change lanes, climbing lanes, turning lanes).

##### B. Urban Arterials, Urban Collectors, and Local Urban Streets

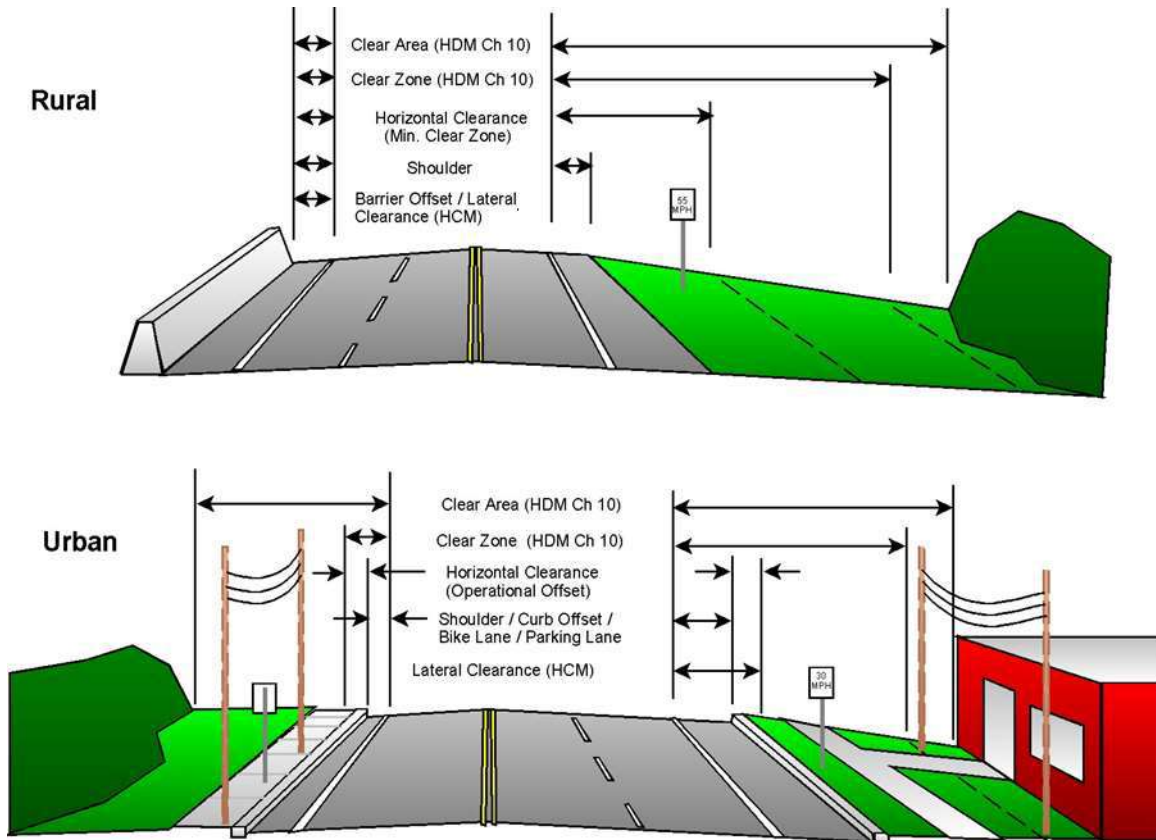
Horizontal clearance functions as an “operational” offset that minimizes restrictions to traffic flow and provides space for opening car doors, the lateral clearance affecting capacity and vehicle position within a lane, and vehicle overhangs at intersections. The area within the horizontal clearance is to be an unobstructed, relatively flat area provided beyond the edge of traveled way. Obstructions include sign posts, lighting posts, poles, hydrants, trees, bollards, etc. The width is measured from the face of curb.

##### C. Turning Roadways

Along turning roadways, horizontal clearance functions as a portion of the clear zone that minimizes restrictions to traffic flow and provides space for the lateral clearance and vehicle position within a lane, disabled vehicles, and vehicle overhangs during turning movements. The area within the horizontal clearance is to be an unobstructed, relatively flat area provided beyond the edge of traveled way. Obstructions include sign posts, lighting posts, poles, hydrants, trees, bollards, etc. The width is measured from the edge of traveled way.



## Exhibit 5-19 Horizontal Clearance Road Sections



Note: Terms based on AASHTO's *A Policy on Geometric Design of Highways and Streets*, unless otherwise noted.



**Exhibit 5-20 Minimum Horizontal Clearances**

Classification	Minimum Horizontal Clearance <sup>1</sup>		Exceptions
	With Barrier	Without Barrier	
<b>Interstates and Other Freeways</b>	Greater of the shoulder width or 4 ft.	15 ft.	<ul style="list-style-type: none"> <li>On bridges where NYSDOT Bridge Manual, Section 2, allows less than 4 ft.</li> <li>In depressed sections, where the minimum is the shoulder width plus 2 ft.</li> </ul>
<b>Rural Arterials</b>	Greater of the shoulder width or 4 ft.	10 ft.	<ul style="list-style-type: none"> <li>On bridges where NYSDOT Bridge Manual, Section 2, allows less than 4 ft.</li> </ul>
<b>Urban Arterials</b>	0 ft. <sup>2</sup>	1.5 ft <sup>2</sup>	<ul style="list-style-type: none"> <li>3 ft. min. at intersections<sup>2</sup></li> </ul>
<b>Rural Collectors</b>	Greater of the shoulder width or 4 ft.	10 ft.	<ul style="list-style-type: none"> <li>On bridges where NYSDOT Bridge Manual, Section 2, allows less than 4 ft.</li> </ul>
<b>Urban Collectors</b>	0 ft. <sup>2</sup>	1.5 ft <sup>2</sup>	<ul style="list-style-type: none"> <li>3 ft. min. at intersections<sup>2</sup></li> </ul>
<b>Local Rural Roads</b>	Greater of shoulder width or 4 ft.,	6 ft. for low-speed (45 mph) segments 10 ft. for high-speed (50 mph) segments	<ul style="list-style-type: none"> <li>On bridges where NYSDOT Bridge Manual, Section 2, allows less than 4 ft.</li> </ul>
<b>Local Urban Streets</b>	0 ft. <sup>2</sup>	1.5 ft <sup>2</sup>	<ul style="list-style-type: none"> <li>3 ft. min. at intersections<sup>2</sup></li> </ul>
<b>Turning Roadways (Ramps)</b>	Right side - greater of shoulder width or 6 ft. Left side - 3 ft. minimum.		<ul style="list-style-type: none"> <li>Where ramps pass under structures, there should be an additional 4 ft. clearance beyond the outside of shoulders to bridge piers or abutments.</li> </ul>

**Notes**

1. Measured from edge of traveled way to obstructions, unless otherwise noted.
2. Measured from face of curb to obstructions.

#### 5.7.12.2 Railroad Clearances

The minimum vertical clearance over the operating mainline tracks of a railroad is generally 22 ft (6.71 m). Other clearances may be justified on some occasions. Refer to [Chapter 23](#) of this manual and Section 2 of the Department's [Bridge Manual](#). The Structures Division will provide guidance.

#### 5.7.12.3 Waterway Clearances

The Department's policy on the minimum vertical clearance over streams and navigable waterways is contained in Section 2 of the Department's [Bridge Manual](#). The Structures Division will provide guidance on specific projects.

The United States Coast Guard shall be contacted to determine the required horizontal clearance along navigable waterways.

#### 5.7.12.4 Airway Clearances

Whenever a project is proposed within 2 miles (3.2 km) of an airport or heliport, the vertical and horizontal clearance between the highway and the airway must be considered. The guidelines for these clearances are contained in Part 75 (Approval of Private ly Owned Airports) of Title 17 of the *Official Compilation of Codes, Rules and Regulations of the State of New York*. The Aviation Division in the Main Office and the Regional Aviation Liaison will provide assistance. The Aviation Division in the Main Office must be notified of any possible conflicts.

The administrator of the airport or heliport should be contacted to determine the facility's long range plans. Any planned changes in the operation of the facility should be considered in the development of the plan and profile of the highway.

A permit from the Federal Aviation Administration (FAA) may be required for vertical elements such as signs and light poles. Warning lights may be required. More information on obstruction evaluation and permitting is available on the FAA website at <https://oeaaa.faa.gov/oeaaa/external/portal.jsp>.

### 5.7.13 Rollover

Rollover is the measure of the difference in cross slope between two adjacent highway lanes or a highway lane and its adjacent shoulder. Maximum rollover rates are shown in Exhibit 5-21, below.

**Exhibit 5-21 Maximum Rollover Rates**

Classification	Between Travel Lanes	At Edge of Traveled Way <sup>1</sup>
Interstates & Other Freeways	4% max.	8% max.
Rural Arterials		
Rural Collectors		
Local Roads & Streets		
Turning Roadways (Ramps)		
Urban Arterials	4% max.	8% max.
Urban Collectors		

**Notes**

1. Refer to HDM Chapter 3, Section 3.2.5.1, Shoulder Cross Slopes and Rollover Limitations, for further guidance.

**5.7.14 Bridge Roadway Width**

A bridge is a structure, including supports, erected over a depression or an obstruction such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20'. The bridge roadway width is the clear distance between inside faces of bridge railing, or the clear distance between faces of curbs, whichever is less. The bridge roadway width includes travel lanes, areas flush with the travel lanes (turn lanes, flush medians, shoulders, curb offsets, parking lanes, and bike lanes), and the Department's standard 5" wide brush curb introduced at the bridge. Bike paths, sidewalks, safety walks, and curbing for sidewalks or safety walks are not part of the bridge roadway width.

Bridge roadway widths should be determined from NYSDOT [Bridge Manual](#), Section 2.

**5.7.15 Passive Snow Control**

Passive snow control involves the mitigation of blowing and drifting snow conditions on roadways through the installation of engineered snow fence, planting of shelterbelts, or by design of an aerodynamic roadway cross section. Use of passive snow control techniques will improve roadway safety by reducing whiteouts and drifting. Removal of snow by mechanical means is approximately 100 times more expensive than trapping snow by passive control. Passive snow control measures should be considered where implementation is feasible and cost-effective.

More detailed information on the design and installation of passive snow control measures can be found in References 7 and 17 in §5.10 of this chapter.

#### 5.7.15.1 Snow Fences

Snow fences may be permanent or temporary. Permanent fences erected on private property will require the acquisition of a permanent easement. Temporary fences may be erected on private property under Article 3, Section 45 of the Highway Law. Some important factors for designing snow fences are:

- The most important factor in designing a snow fence is capacity. Fences should be of adequate height to store the average annual amount of snow that will be transported (blown) through the problem area.
- To maximize effectiveness, fences should be at least 8 ft (2.4 m) in height, but may be as short as 6 ft (2 m) in areas of limited snow transport or restricted right of way.
- Fences should be located at least 35 times their effective height (total height minus ambient snow depth) from the road shoulder.
- A single row of tall fences is preferable to multiple rows of shorter fences.
- Fences should be perpendicular to prevailing wind directions but departures up to 25° are acceptable. They should be placed parallel to the roadway whenever possible.
- A gap equal to 10% of the total fence height should be left under the fence to reduce snow deposition near the fence. Deposition at the fence reduces the fence's efficiency.

#### 5.7.15.2 Shelterbelts

Also referred to as "living snow fences," shelterbelts are multiple rows of trees, preservation of agricultural crops, or shrubs planted to provide protection from wind-driven snow. There are many advantages to shelterbelts as compared to snow fences, including roadside beautification, wildlife benefits, little or no maintenance after establishment, and long service life. The Regional Landscape Architect and Maintenance Environmental Coordinator should be consulted whenever shelterbelts are considered.

Some design tips for planting shelterbelts are:

- Trees should be placed no closer than 3 times their mature height from the edge of shoulder.
- Generally, trees should be coniferous. Shrubs may be effective in areas of limited blowing and drifting snow.
- Two or more staggered rows should be planted to provide full coverage and to prevent gaps caused by plant loss or damage.
- Trees should be spaced so that crown closure will be achieved within ten years.
- Shrubs may be used where cost or space limitations do not allow for the planting of trees.

- An effective shelterbelt can be achieved by requesting farmers to leave six to eight rows of corn stalks standing through the winter. The minimum setback from the road shoulder should be 35 times the effective stalk height (height minus ambient snow depth).

#### 5.7.15.3 Drift-Free Roads

Providing an aerodynamic roadway cross section will allow the roadway to be swept clear by the wind. It should be recognized that this is generally not a good solution where whiteouts are a problem. However, there may be some instances where the existing cross section may be contributing to the visibility problem and roadway redesign will be a viable alternative to mitigate the problem.

In some areas, it may be possible to reduce drifting by altering the cross section to provide for additional snow storage upwind from the road. Minor grading on private property may be accomplished with a property release from the owner.

The following guidelines, implemented individually or in combination, as appropriate, will improve drift-prone areas:

- Backslopes and foreslopes should be flattened to a 1:6 slope or flatter.
- Ditches should be widened as much as possible.
- The profile of the road should be raised to 2 ft (0.6 m) above the ambient snow cover.
- Provide a ditch that is adequate for storing the snow plowed off the road
- Widen cuts to allow for increased snow storage.
- Eliminate the need for guide rail.
- If existing W-beam guide rail appears to be contributing to a drifting problem, consideration should be given to replacing the W-beam with cable or box beam guide rail.

### 5.7.16 Parking

#### 5.7.16.1 On-Street Parking

On-street parking spaces in village and urban settings complete for usable streetscaping and pedestrian space. Early contact with local officials and business owners is important to identify acceptable locations for parking spaces.

On-street parking adds an element of traffic calming whereby drivers are inclined to lower their speeds when confronted with slow-moving drivers looking for a parking space, or drivers of parked vehicles opening their doors. However, care must be taken in introducing any new on-street parking. Since high-speed traffic is not compatible with slow-moving vehicles and limited sight distances, on-street parking is not to be added to facilities with

design speeds of 50 mph (80 km/h) or more. When adding on-street parking to low-speed facilities, the designer should consider:

- The impact on the highway's safety and capacity.
- Snow storage and removal.
- Pedestrian visibility and access to sidewalks and crosswalks. Midblock crossings and curb bulb-outs to help motorists anticipate and see pedestrians among the parked vehicles. See Chapter 18 of this manual for additional guidance and recommendations to maintain visibility and reduce conflicts.
- Parked vehicles can block emergency vehicles, such as fire trucks, from direct access to buildings.
- Bicycle facility presence and use. See Chapter 17 of this manual for additional guidance and recommendations to minimize conflicts.

To provide for adequate sight distance, vehicle turning paths, and emergency vehicles, parking is to be restricted near an intersection, fire house, commercial driveway, railroad crossing, fire hydrant, safety zone, pedestrian crosswalk, etc. As an exception, parking may be permissible opposite a minor T-intersections along a low speed highway. Refer to §1202 of the NYS Vehicle & Traffic Law for additional guidance (Note that §1621 of the NYS Vehicle & Traffic Law allows the Department to establish other distances).

#### A. Parallel Parking

On local roads and streets, collector roads and streets, and arterials, parallel parking is a design consideration due to land-use patterns and a lack of off-street parking facilities. [Chapter 2](#) of this manual presents minimum parking lane widths for functional classifications of highways. Parking stalls should be 22 ft (6.6 m) to 25 ft (7.8 m) long. Parking lanes normally are not carried across bridges unless the bridge is less than 50 ft (15 m) in length, in which case it may be considered.

#### B. Diagonal (Angled) Parking

Front-in diagonal parking is to be avoided due to restricted driver visibility while backing out of the parking space into traffic, including cyclists and motor cyclists. Where this type of parking exists, it should generally be eliminated by providing parallel parking or back-in diagonal parking in low-speed areas, and off-street facilities in high-speed areas.

Back-in diagonal parking allows motorists to back into parking stalls, similar to backing into parallel spaces, while retaining the greater parking density of diagonal spaces. Backing into the space is no more disruptive to traffic than parallel parking. Compared to front-in diagonal parking, back-in diagonal parking places the motorist in a much better position to view motor vehicle and bicycle traffic when pulling out of the parking stall. It also makes it safer to load groceries and other items into the rear of the vehicle. Back-in diagonal parking has been successful in Canada, Washington State, and other areas.

In instances where other parking measures are not feasible and no related crash experience exists, front-in diagonal parking may be retained on local streets design speeds are 30 mph (50 km/h) or less and traffic volumes are low.

Diagonal parking stalls should be 8 ft to 9 ft (2.5 m to 2.7 m) wide and 17 ft to 18 ft (5.2 m to 5.5 m) long. Although wheel stops are desirable to prevent parked vehicles from encroaching into the sidewalk area, they should not be installed since they will interfere with snow removal operations. A snow storage area may be used to prevent parked vehicles from encroaching into the sidewalk area.

### C. On-Street Parking Requirements for Persons with Disabilities

Parking requirements for persons with disabilities requires special consideration, and some requirements that are mandatory. Refer to [Chapter 18](#) of this manual, for the standards for accessible parking in accordance with the "Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way" (PROWAG), the Vehicle and Traffic Law, and the State Building Code.

In general, accessible on-street parking should be located close to key destinations, and where the street has the least crown and grade. The sidewalk adjacent to accessible on-street parking spaces should be free of obstructions that may prevent the deployment of a van side-lift or ramp, or may prevent a vehicle occupant from transferring to a wheelchair.

The Regional Landscape Architect can provide additional advice regarding the location and design of accessible on-street parking spaces.

#### 5.7.16.2 Off-Street Parking

When on-street parking spaces are eliminated to improve traffic operation and safety, replacement off-street parking should be considered in accordance with Section 10-40 of the Highway Law. The off-street lots should be located as close as possible to the eliminated on-street spaces to provide adequate access to properties formerly served by the on-street spaces. The number of off-street spaces provided should approximate the numbers of eliminated on-street spaces unless a parking utilization study indicates otherwise.

#### A. Off-Street Parking for Persons Who Are Not Disabled

- Stall widths of 9.5 ft or 10 ft (2.9 m or 3.0 m) should be provided for lots with short-duration, high-turnover parking and for lots serving customers with packages.
- Stall widths of 8.5 ft or 9 ft (2.6 m or 2.7 m) should be used for lots with longer duration, low-turnover parking.

#### B. Off-Street Accessible Parking

All off-street accessible parking spaces must also comply with the provisions of the NYS Uniform Fire Protection and Building Code and as required by the NYS Vehicle and Traffic Law. Additionally, off-street accessible parking spaces must conform to the requirements

of the "Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way " (PROWAG), [Chapter 18](#) of this manual, and Standard Sheet 608-02.

The Regional Landscape Architect can provide additional advice regarding the location and design of accessible on-street parking spaces.

The Regional Landscape Architect and Regional Transportation Systems Operations Engineer should be contacted for further information on design of off-street parking lots including stall depths, aisle widths, and other layout dimensions and features such as provisions for persons with disabilities, landscaping, and lighting ( refer to [Chapter 12](#) of this manual).

### 5.7.17 Access Control

Access control is the regulation of public access to and from properties and roads abutting highway facilities to preserve safety and capacity. These regulations are categorized as full control of access, partial control of access, and driveway or intersection approach regulations.

Full control of access gives preference to through traffic by providing access connections only with selected public roads utilizing interchanges. Criteria for interstate highways and other freeways are presented in [Chapter 2](#) of this manual. Interchange access control principles are presented in [Chapter 6](#) of this manual.

Partial control of access gives preference to through traffic to a degree that, in addition to access connections with selected public roads utilizing interchanges, there may be some at -grade intersections and/or driveway connections.

Driveway and intersection approach regulations allow access to and from all abutting properties and streets in a controlled manner.

Access control should be included in the design of all highways, especially where the likelihood of commercial development exists. The extent of control should be coordinated with the local land- use plan to ensure that the desired degree of control can be maintained through local zoning ordinances. Access management will enhance highway safety and minimize vehicle delays.

#### 5.7.17.1 Interstates and Other Freeways

Access to the interstate system shall be fully controlled. Access is to be achieved by interchanges at selected public highways. Access control shall extend the full length of ramps and terminals on the crossroad. Such control shall either be acquired outright prior to construction or by the construction of frontage roads or by a combination of both.

Control for connections to the crossroad should be provided beyond the ramp terminals by purchasing access rights or providing frontage roads. Such control should extend beyond the ramp terminal at least 100 ft. in urban areas and 300 ft. in rural areas (refer to [Chapter 6](#) of this manual for more specific details).



The interstate highway shall be grade separated at all railroad crossings and selected public crossroads. All at-grade intersections of public highways shall be eliminated. To accomplish this, the connecting roads are to be terminated, rerouted, or intercepted by frontage roads. Refer to Exhibit 5-22 for a key to highway access issues.

## Exhibit 5-22 Highway Access Issues

Access Issues	Department Requirements, Guidance, and Procedures	Law, Regulations, and Policy
General Access Management	<ul style="list-style-type: none"> <li>Best Practices on Arterial Management, 1997</li> <li><a href="#">HDM Ch 3</a>, § 3.2.8 - Medians</li> </ul>	AASHTO's <i>A Policy on Geometric Design of Highways and Streets</i> , adopted as standards by FHWA for the NHS
ROW Acquisition and Management	<ul style="list-style-type: none"> <li>HDM Ch 5, § 5.5.6 - Types of ROW and Access</li> <li>Real Estate Manual/Directives - Nonmotor vehicle access (e.g., locked gate access to billboards) and relinquishment of ROW</li> <li>Highway Work Permit Process<sup>1</sup></li> </ul>	<ul style="list-style-type: none"> <li>23 CFR 620B "Relinquishment of Highway Facilities"</li> <li>23 CFR 713C "Disposal of Rights-of-Way."</li> </ul>
Accommodation of Utilities	<ul style="list-style-type: none"> <li><a href="#">HDM Ch 13</a> - Utilities</li> <li>Highway Work Permit Process<sup>1</sup></li> </ul>	<ul style="list-style-type: none"> <li>17 NYCRR Part 131 "Accommodation of Utilities within State Highway Right-of-Way."</li> <li>AASHTO's Utility Guide(s)</li> <li>23 CFR 645 B - Accommodation of Utilities</li> <li>Accommodation Plan for Longitudinal Use of Freeway Right of Way by Utilities, 1995</li> <li>Requirements for the Design and Construction of Underground Utility Installations Within the State Highway Right of Way</li> <li>Section 52 of the NYS Highway Law "Permits for work within State highway right of way."</li> </ul>
Access Control Limits	<ul style="list-style-type: none"> <li><a href="#">HDM Ch 2</a>, § 2.7 - Standards</li> <li>HDM Ch 5, § 5.7.17 - Access Control</li> <li><a href="#">HDM Ch 6</a>, § 6.2 - Control of Access</li> <li><a href="#">HDM Ch 3</a>, § 3.2.9.3.C - Access Control</li> <li>Highway Work Permit Process<sup>1</sup></li> </ul>	<ul style="list-style-type: none"> <li>AASHTO's <i>A Policy on Design Standards - Interstate System</i></li> <li>AASHTO's <i>A Policy on Geometric Design of Highways and Streets</i>, adopted as standards by FHWA for the Interstate and NHS, respectively</li> </ul>
Driveways	<ul style="list-style-type: none"> <li>HDM Ch 5, § 5.7.18 - Driveways</li> <li>HDM Ch 5, Appendix A, NYSDOT's "Policy and Standards for Design of Entrances to State Highways" (a.k.a. Driveway Design Policy)</li> <li>Highway Work Permit Process<sup>1</sup></li> </ul>	<ul style="list-style-type: none"> <li>Section 52 of the NYS Highway Law "Permits for work within State highway right of way."</li> <li>17 NYCRR Part 125 "Entrances to State Highways."</li> </ul>
Freeway Breaks in Access	<ul style="list-style-type: none"> <li><a href="#">PDM Appendix 8</a> - Freeway Access Modification Procedures</li> <li>Highway Work Permit Process<sup>1</sup></li> </ul>	FHWA <a href="#">Policy on Access to the Interstate System</a> , May 22, 2017

**Note:**

1. The Highway Work Permit Process is for work not progressed by Department projects or maintenance forces.

### 5.7.18 Driveways

Where driveways are allowed for access to and from the highway, they are to be designed in conformance with the latest edition of the Department's "Policy and Standards for Design of Entrances to State Highways" included as Appendix A of this chapter. When curb is used for driveway control, it shall be consistent with the guidance and requirements of Chapters [3](#) and [10](#) of this manual.

To obtain adequate geometrics for driveway entrances, it may be necessary to extend the limit of work beyond the existing highway boundary. Section 5.5.6.6 discusses releases for "Reestablishment of Approaches to Private Lands" to be used for this work.

### 5.7.19 Frontage Roads - Service Roads

Frontage roads are partially or uncontrolled access highways parallel to controlled access highways. Frontage roads:

- Provide access to the adjoining property and local traffic circulation.
- Segregate lower speed local traffic from higher speed through traffic.
- Help preserve the safety and capacity of the controlled access highway by reducing or eliminating access points to the through highway.

The design criteria of the frontage road should be based upon its functional classification. See [Chapter 2](#), Section 2.7.5.7, of this manual. Frontage roads are generally local roads or streets. In most cases, they should be turned over to the local unit of government for maintenance.

For further discussion of frontage roads, see Chapter 4 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

### 5.7.20 High-Occupancy Vehicle (HOV) Lanes

HOV lanes are travel lanes along freeways and other multilane highways which are designated solely for use by carpools, vanpools, buses, and other vehicles carrying a specified minimum number of people. When operated at a suitable level of service, an HOV lane is more efficient than a conventional-use lane because more people are moved per hour. An HOV lane can be constructed with the capability of being reversed to serve the peak hour direction. HOV lane(s) may provide an alternative solution to existing or projected congestion problems when environmental, fiscal, or policy decisions preclude construction of additional or adequate numbers of conventional lanes.

Drivers may be encouraged to use HOV lanes through incentives such as reduced, reliable travel times compared to adjacent conventional use lanes, special access ramps, reduced tolls, special toll booths, and preferred and/or cheaper parking at the job site. More detailed information including design guidelines for HOV facilities can be found in [Chapter 24](#) of this manual.

### 5.7.21 Complete Streets

#### 5.7.21.1 Transit Bus Stops

Bus stops are generally located where there is concentrated commercial, residential, office, or industrial development or at intersections of arterial or major collector streets. Bus stops can be provided at the far side or the near side of an intersection or at midblock. Whenever possible, bus stops should be located at the far side of intersections to facilitate bus and traffic operations. The transit operator should be consulted for all bus stop placements. Pedestrian design treatments such as placing bus stops at signalized intersections, and providing adequate sight distance for pedestrians should be considered when pedestrians will be required to cross the road.

The curb adjacent to the bus stop should be painted and signs posted to clearly identify the area as no parking or stopping except for buses. Pedestrian facilities should be provided (e.g., sidewalks and wheelchair access ramps). A marked pedestrian crosswalk should be considered if one is not in the immediate vicinity and there are pedestrian generators (e.g., school, commercial areas, residential areas, a sidewalk, a park) on the other side of the street. Refer to [Chapter 18](#) for a discussion of marked crosswalks.

Ideally, bus passenger shelters should be provided at every bus stop. Transit providers should be consulted on shelter design. Design standards must comply with the requirements of the *Proposed Guidelines for Pedestrian Facilities in the Public Right-of-Way (PROWAG)* and [Chapter 18](#) of this manual. [Chapter 24](#), Section 24.3.5, of this manual provides additional information on bus stops.

#### 5.7.21.2 Transit Bus Turnouts

A bus turnout is a bus stop (refer to Section 5.7.21.1) located in a recessed area adjacent to lanes of moving traffic. A turnout should be considered whenever potential for auto/bus conflicts warrants separation of transit and general-purpose vehicles, but especially where a bus stopping in a travel lane may be unsafe or impede traffic flow.

Turnouts must be designed to safely accommodate bus ingress and egress movements and passenger loading and unloading activity. Conflicts with driveways should be avoided for the length of the turnout. The transit operator should be contacted to ensure the turnout will be used by the bus drivers. Refer to Section 5.9.9 of this chapter, [Chapter 24](#), Section 24.3.6, of this manual, and Chapter 4 of AASHTO's *A Policy on Geometric Design of Highway and Streets*, 2018.

#### 5.7.21.3 Transit Bus Turnarounds

Bus turnarounds are facilities that expedite a bus's return to the service route. Turnarounds can be used at the termini of routes to turn transit vehicles or they can be incorporated into a land-use development design.

Turnarounds should be designed to allow a bus to turn in a counter-clockwise direction to improve the driver's visual capabilities. The design should allow adequate space for a bus to pass a standing vehicle. A jug-handle bus turnaround design may be used at midblock bus

terminal locations. "Cul de sac" and loop designs are acceptable for developments that do not have internal roadway networks to return a bus efficiently to the arterial roadway. They should be used only at the end of a bus route. The transit operator should be consulted for all turnaround placements and designs. [Chapter 24](#), Section 24.3.7, of this manual provides additional information on bus turnarounds.

#### 5.7.21.4 Pedestrians

The needs of pedestrians, especially disabled pedestrians, are an important part of the roadway environment in rural as well as in urban areas. Evaluating and meeting the needs for pedestrian accommodations and safety are important considerations during scoping and design. [Chapter 18](#) of this manual provides information on the design of pedestrian facilities. Coordinate with the Regional Bicycle/Pedestrian Coordinator when assessing the need for pedestrian facilities, and the Regional Landscape Architect and Regional ADA Specialist for assistance in designing appropriate, accessible facilities.

#### 5.7.21.5 Bicyclists

The accommodation of bicyclists is important as more and more cyclists are utilizing the transportation system recreationally, for commuting, and the delivery of goods and services. The benefits derived from relieving congestion, reducing air pollution, lowering energy costs, promoting healthy lifestyles, and contributing to quality communities should not be underestimated. Project developers should coordinate with the Regional Pedestrian/Bicycle Coordinator in assessing the need for bicycle facilities and the Regional Landscape Architect for assistance in designing appropriate facilities.

A discussion on necessary provisions for bicyclists and specific design standards is included in [Chapter 17](#) of this manual.

## 5.8 DESIGN CONSIDERATIONS

### 5.8.1 Driver Expectancy

Drivers expect and anticipate certain geometric and operational characteristics along a roadway. Roadway features that violate driver expectancy have a direct influence on safety. Drivers, particularly those unfamiliar with or inattentive to their surroundings, can be lulled into complacency and may react inappropriately when confronted with unexpected changes. To reduce potential collisions, designers should maintain consistency throughout a highway segment and gradually transition from one segment to another. Gradual transitions notify and prepare the driver for upcoming changes. When gradual transitions are not practical, warning signs, lighting, flashing warning lights, and additional sight distance should be considered.

Some typical features to avoid are:

- Sharp horizontal curves (i.e., those curves requiring a design speed drop of 10 mph [15 km/h] or more following long tangents).
- Upgrading alignment without corresponding cross section improvements. (This can cause an erroneous and possibly a false illusion of improved safety, which may encourage operating speeds that are excessive for the pavement width and clear zone.)
- Compound curves – Refer to Section 5.7.3.5.A
- Broken back curves – Refer to Section 5.7.3.5.B

### 5.8.2 Geometric Considerations

#### 5.8.2.1 Combination of Horizontal and Vertical Alignment

Horizontal and vertical alignments should not be designed independently. They complement each other and their interrelationship can have a significant effect on the operational and safety characteristics of a section of roadway. Proper combinations of horizontal alignment and profile should be determined by engineering study and consideration of the general controls listed in Section 3.5 of AASHTO's, *A Policy on Geometric Design of Highways and Streets*, 2018.

#### 5.8.2.2 Right of Way Impacts

The effects of land acquisition must be considered in every stage of a project's development. Although it is of primary importance to meet appropriate standards for the selected highway, designers must not be so concerned with traffic data and standards that they neglect entirely the value of local culture and the natural beauty of the land traversed. Much controversy can be avoided by knowing what features are considered important by the people who live in the project area and by designing to minimize a project's impact on those features without compromising safety.

Some principles to keep in mind when designing projects are:

- In rural areas, shallow fill sections generally require less right of way than cut sections because they reduce the amount of roadside ditching. (Shallow fill sections are also less susceptible to blowing and drifting snow problems than cut sections.)
- Although right of way impacts cannot always be avoided, many times they can be reduced through creative design efforts, particularly in sensitive areas. The use of centerline shifts, special ditches, lawn pipes, tip-up shoulders, field inlets, and gabions are just a few of the techniques which could be considered when investigating alternative designs.
- The cost of right of way is an important factor to consider when investigating project alternates. Right of way cost should be balanced against construction and future maintenance costs.

#### 5.8.2.3 Balancing Cuts and Fills

When developing a roadway profile, it is generally cost-effective to provide a balance between cut and fill sections. However, the need to minimize impacts to adjacent properties often overrides the benefit of a highway with balanced cuts and fills.

### 5.8.3 Joint Use of the Highway Corridor

The joint use of transportation corridors is a legitimate and necessary utilization of right of way. The accommodation of pedestrians, bicyclists, and utilities, along with transit, commercial, and private motor vehicles, provides a more comprehensive transportation system resulting in significant cost, mobility, and environmental benefits. The designer must recognize the positive implications of this sharing of the transportation corridor and consider not only the safe, efficient movement of vehicles, but also, the movement of people, the distribution of goods, and the provision of essential services.

### 5.8.4 Social, Economic, and Environmental Considerations

Throughout the project development process, consideration must be given to mitigation for areas impacted by Department projects. Mitigation is defined by the Council on Environmental Quality as avoiding, minimizing, rectifying, reducing/eliminating, and compensating for impacts. Detailed information is provided in NYSDOT's [Project Development Manual](#) and [Environmental Manual](#).

### 5.8.5 Utilities

Utility facilities occupy State highway right of way either by law or by permission of the Department. Utility occupation is subordinate and subject to the use of the right of way by the Department for highway or other purposes authorized by law. It is in the public interest for utilities to be accommodated within the highway right of way.

Utilities must be considered when scoping and designing a project. Early coordination between the Department and utilities is vital to a successful design. With early and reliable utility as-built information, many costly relocations can often be reduced or eliminated through minor design changes. While it may be necessary for designers to make adjustments for utility accommodations, acceptable safety standards must always be maintained.

Some types of utility relocations are eligible for reimbursement by the Department, but the Department does not subsidize utility accommodations. For additional information refer to Part 131 (*Accommodation of Utilities Within State Highway Right-of-Way*) of Title 17 of the "Official Compilation of Codes, Rules, and Regulations of the State of New York" and Chapters [10](#) and [13](#) of this manual.

### 5.8.6 Increasing Capacity Without Adding Lanes

As congestion increases and there is less opportunity to provide additional lanes, there are a number of options that can and should be considered when designing a project to relieve congestion. The following subsections briefly describe some of the measures that may be used. Mobility measures are described in more detail in [Chapter 24](#) of this manual. The design effort for these measures should be coordinated with the Regional Planning and Program Manager and the Regional Transportation Systems Operations Group.

#### 5.8.6.1 Intelligent Transportation Systems (ITS)

Applying the technologies of advanced communications, information processing, sensing systems, and computer control systems to control vehicles operating on the highway and transportation network is known as ITS. Employing ITS strategies can improve the operation of the existing transportation system by redirecting traffic to avoid congestion, providing assistance to drivers and other travelers on planning and following optimal routes, increasing the reliability of and access to information on public transportation routes and schedules, and refocusing safety efforts on crash avoidance rather than just minimizing the consequences of crashes. It will include such strategies as rapid response to road crashes to restore traffic flow, changeable message signs to inform drivers of current road conditions, better information on ridesharing opportunities, control of traffic movement, route guidance systems, and electronic toll collection, to name a few.

#### 5.8.6.2 Ramp Metering

Ramp meters are considered to be a very cost-effective technique for improving traffic flow on freeways, protecting mainline capacities and improving overall operational efficiencies during peak flow periods. A ramp meter is a modified traffic signal which is located on a ramp and which operates at a controlled rate to regulate the flow of traffic from the ramp to the



freeway. The rate at which vehicles are released into the freeway lane is based on the freeway traffic volume. When congestion on the freeway is heavy, the release rate is less frequent. During off-peak periods, the signal may revert to pretimed intervals or may be taken out of service until the next period of congestion. Further information can be found in [Chapter 24](#), Section 24.5.1, of this manual.

#### 5.8.6.3 Reversible-Flow Traffic Lanes

When the peak travel demand on a multiple lane facility is significantly greater in one direction than in the other and that demand is reversed between the morning and evening periods, reversible-flow operation may be justified. It can be applied to mixed-use traffic on undivided or divided urban arterials and to express buses or other HOVs on arterials or freeways. Reversible-flow lanes are usually located in the center or median lane(s). During off -peak periods the operation on arterials can revert back to the normal traffic pattern. It is generally desirable to separate reversible-flow traffic lanes from the mixed-use traffic by physical barriers. In addition, the lanes to be reversed and the direction of traffic flow can be designated by specific traffic signals suspended over each lane and by permanent signs advising motorists of changes in traffic regulations and the hours they are in effect. Further information on reversible-flow lanes may be found in [Chapter 24](#), Section 24.2.4, of this manual.

#### 5.8.6.4 Shoulders as Travel Lanes on Freeways

On a freeway that requires increased capacity due to congestion, converting the existing shoulder to a travel lane may be the most expedient and economical method of adding capacity when compared to the alternative of purchasing additional right of way and adding a new lane. It may be done, for example, when queues develop at freeway-to-freeway interchanges, or when congestion occurs at bottlenecks or merge points, or when peak periods exceed 3 hours in duration. Care should be taken to ensure that the resultant loss of the shoulder(s) does not produce more congestion-related problems and crashes than it eliminates. Driving on the shoulders of state-controlled access highways is prohibited and must be authorized by the Department. Further discussion of the use of freeway shoulders as peak-hour travel lanes is found in [Chapter 24](#), Section 24.5.4, of this manual.

#### 5.8.6.5 Priority Treatment for HOVs on Arterials

Providing priority treatments for buses at traffic signals on arterial streets has the potential for reducing delays, in effect, increasing capacity. For example, turn restrictions are often used to increase capacity where limited space prevents the addition of a lane or lanes. The turn restrictions may disrupt bus routes and schedules by forcing them to travel greater distances. By exempting buses from the turning restrictions, distances and travel time can be reduced, reducing delays to the greater number of bus passengers. Passive systems for granting priority to HOVs involve signal timing adjustments to favor the direction of flow with the greater number of HOVs, or providing special HOV phases on facilities with reserved lanes or streets. Active systems include special equipment for buses which allow preemption of normal traffic signal cycles. Further information on priority treatment systems can be found in [Chapter 24](#), Section 24.5.8, of this manual.

#### 5.8.6.6 Upgrading the Signal System from Pretimed to Actuated Control

Actuated signal systems adjust the signal timing based on vehicle detection systems. Vehicle detection systems are described in [Chapter 11](#), Section 11.3.2, of this manual.

#### 5.8.6.7 Coordinated Signal System

Coordinated signal systems are two or more synchronized signals that allow vehicles to travel through each signal without stopping. Coordinated signal systems are described in [Chapter 11](#), Section 11.3.3.5F, of this manual.

#### 5.8.6.8 Elimination of On-Street Parking

Elimination of on street parking can reduce the delay caused by traffic slowing while vehicles enter and exit the parking lane. The parking lane can help provide space for turn lanes and/or a median to prevent mid-block left turns. Refer to Section 5.7.16 for a discussion of off-street parking areas.

#### 5.8.6.9 Eliminating Mid-Block Left Turns

The installation of raised medians can improve capacity and safety on uncontrolled access facilities by eliminating mid-block left turn maneuvers. Roundabouts, U-turns, jug handles, or indirect lefts can help provide access for those who would otherwise make a left turn. Refer to Section 5.9.1 for a discussion of these intersection configurations.

#### 5.8.6.10 Converting a 2 or 4 Lane Section with Shoulders to a 3 or 5 Lane Section

Where ROW is severely constrained, a two-way left-turn lane (TWLTL) may provide more overall benefit to safety and capacity than wide shoulders depending on the travel speeds, traffic volume, turning volumes, frequency of commercial driveways, and crash history. Refer to Section 5.9.8.2.C for a discussion of TWLTLs.

### 5.8.7 Traffic Calming

Traffic calming recognizes the significance of sharing the transportation corridor by employing techniques to reduce vehicle speeds and volumes. Traffic calming measures can help support the livability and vitality of a residential or commercial area through an improvement in non-motorist safety, mobility, and comfort. Examples of measures include slow points, street closures, and narrow and short streets. Further information on traffic calming can be found in [Chapter 25](#) of this manual.

### 5.8.8 Aesthetics

The visual quality of travel corridors should be considered along with the safe, efficient movement of people and goods. The lands adjacent to highways are the most visible to the traveling public, often defining the image and character of the locale and region.

The creation or preservation of an attractive landscape can contribute to safety, environmental, and economic benefits. The careful blending of the highway with the natural and cultural landscape, the careful grading of cut and fill slopes, and the selective preservation of vegetation, supplemented with new plantings where appropriate, help to:

- Define the highway for the motorist.
- Reduce the potential for erosion.
- Reduce the need for precautionary signing and guide rail.
- Reduce construction costs.
- Integrate the project into the surrounding area.

To minimize the visual impact from adjacent sensitive viewing locations, particularly along high-volume roadways, the feasibility of screening the highway should be investigated during design. Project developers should coordinate with the Regional Landscape Architect in assessing the project needs and in designing appropriate measures.

## 5.9 INTERSECTIONS AT GRADE

Highway crossings may be grade separated or at-grade (signalized and unsignalized). Grade-separated crossings do not provide access between the crossing highways unless an interchange is constructed. Interchanges consist of special purpose roadways (ramps) which provide either partial or complete access between the highways. The decision whether to provide an at-grade or a grade-separated highway crossing is a trade-off between providing optimal service to through traffic on one or both highways and providing access to surrounding land uses and should be based on the highway functional classification and operational and safety considerations. The type of crossing selected should meet capacity, safety, and mobility needs and be consistent with Regional land use plans. Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, provides guidance on the selection of a type of crossing.

Intersections influence and, in some cases, are a prime determinant of operating conditions on each intersecting highway and consequently merit special consideration in design. In urban or suburban areas, crashes and capacity constraints are often concentrated at intersections.

Intersections should provide access between highway approaches at a level of service consistent with driver expectations for the highway, incorporate cost effective mitigation of crash patterns on existing facilities and address safety issues on new facilities.

Design of intersections should be consistent with the design considerations and recommendations contained in [Chapter 18](#) of this manual and Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

### 5.9.1 Types of Intersections

The basic at grade intersection types are the circular, angular, and nontraditional intersections. Circular intersections include the traffic circle, rotary, and roundabout. Angular intersections include three-leg, such as T- or Y- intersections, four-leg, and the multileg. Nontraditional intersections include the Super-Street Median Crossover and Continuous Flow Intersection. Operational considerations for selecting an intersection layout include design-hour volumes and predominant movements, types and mix of vehicles, pedestrians, bicyclists, approach speeds, number of approaches, and safety needs. Local conditions and right of way costs often influence the intersection that is feasible and the associated design elements. The alignment and grade of intersecting highways combined with crash patterns may require channelizing the intersection regardless of the traffic volumes (refer to Section 5.9.4 of this Chapter).

General objectives for intersection design are:

- To provide adequate sight distances.
- To minimize points of conflict.
- To simplify conflict areas.
- To limit conflict frequency.
- To minimize severity of conflicts.
- To minimize delay.
- To provide acceptable capacity for the design year.

Roundabouts are frequently able to address the above objectives better than other intersection types in both urban and rural environments and on high- and low-speed highways. Thus, when

a project includes reconstructing or constructing new intersections, a roundabout alternative is to be analyzed to determine if it is a feasible solution based on site constraints, including ROW, environmental factors, and other design constraints. Exceptions to this requirement are where the intersection:

- Has no current or anticipated safety, capacity, or other operational problems.
- Is within a well working coordinated signal system posted at under 45 mph with acceptable crash histories.
- Is where signals will be installed solely for emergency vehicle preemption.
- Has steep terrain that makes providing an area, graded at 4% or less for the circulating roadways, infeasible.

When the analysis shows that a single lane roundabout is a reasonable alternative, it should be considered the Department's preferred alternative due to the proven substantial safety benefits and other operational benefits. Guidance on roundabouts can be found in Chapter 26 of this manual.

**Note:** A reasonable alternative is a feasible solution that meets the objectives in a cost-effective and environmentally sound manner. The preferred alternative is the reasonable alternative that the Department is leaning toward recommending for design approval. The preferred alternative can change if a new reasonable alternative is identified and as the reasonable alternatives are evaluated during preliminary design.

#### 5.9.1.1 Circular Intersections

Traffic circles and rotaries, popular during the first half of the 20<sup>th</sup> century, typically have serious operational and safety problems, which include the tendency to lock -up at higher volumes. These intersection types are not to be constructed and should be evaluated for reconstruction when included in a multicourse resurfacing ( i.e., 2R/3R project) or reconstruction project.

Roundabouts offer unique solutions to traffic operations and safety problems at intersections. Generally, for the same traffic volume, delays are less at roundabouts as compared to other controlled intersections (typical delay reductions are 30 -70%). Roundabouts will accommodate large volumes of left turn movements with less delay than signalized intersections. If left turns are minimal, or most of the traffic is making similar moves (i.e., there is a significantly dominant direction of traffic), then a conventional controlled intersection may offer less vehicular delay. With regard to safety, roundabouts reduce vehicle speeds and result in significantly fewer crashes. A study by the Insurance Institute for Highway Safety found that construction of roundabouts resulted in a 39% overall reduction in crashes; a 76% reduction in injury crashes and an 89% reduction in fatal or incapacitating crashes. No other intersection type has been found to provide that magnitude of safety improvement. Refer to Exhibit 5-23 for examples of a one-lane roundabout, atwo-lane roundabout, and a roundabout corridor.

Designers should refer to the roundabout pages on the Department's Internet and IntraDOT sites for the latest requirements, guidance, and public involvement materials for roundabouts. Additionally, designers should contact their Regional expert or the Intersection Design Squads

in the Design Bureau for guidance and assistance throughout the development of the roundabout design.

The initial layout, preliminary plans, and advance detail plans for the roundabout should be reviewed by designers with considerable roundabout design experience. For multi-lane roundabouts, roundabouts with more than 4 legs, and roundabouts with unusual geometry, the Intersection Design Squads should be included in the review by e-mailing the ProjectWise location to [roundabouts@dot.ny.gov](mailto:roundabouts@dot.ny.gov).

#### 5.9.1.2 Angular Intersections

##### A. 3 Legged (T Intersections)

T-intersections are one of the most commonly used intersection types. Refer to Sections 5.9.2 through 5.9.8 of this chapter for guidance and requirements applicable to these intersections.

##### B. Closely Spaced T Intersections

Closely spaced opposing T intersections are offset ("dog leg") intersections, where either the main street or side street approach legs are not aligned with each other. Offset intersections can result in operational problems, depending on the offset distance, traffic control, and the amount of traffic going from one offset leg to the other. Consult with the Regional Transportation Systems Operations Engineer to determine the appropriateness of aligning offset intersection legs.

At offset intersections and divided highway crossings, where the distance between the nearest edges of the two intersecting roadways is 30 ft (9.14 m) or more, two separate intersections exist and must be independently controlled with appropriate intersection control. The Vehicle and Traffic Law definition of roadway is "That portion of a highway improved, designed, marked, or ordinarily used for vehicular travel, exclusive of the shoulder or slope." Refer to Exhibit 5-24.

**Note:** Title I, Article 1, Section 120(b) of the NYS Vehicle and Traffic Law states when two roadways intersect a highway 30 ft (9.14 m) or more apart, each crossing shall be regarded as a separate intersection.

Offset intersection approaches or roadways within 30 ft (9.14 m) of each other may be considered one intersection for the purpose of traffic control. Consult with the Regional Transportation Systems Operations Engineer to determine appropriate traffic control at offset intersections. Coordination of geometric design and traffic control is especially critical at these intersections.

### C. 3 Legged (Y-Intersections)

Y-intersections experience widespread operational and safety problems and should be avoided. If a crash problem is identified, existing Y-intersections should be realigned to T-intersections, replaced by a roundabout, or their retention discussed in the Design Approval Document. The rationale for retention should be based on the lack of a related crash pattern, excessive grade, or unreasonable reconstruction costs and/or impacts. When the angle is 60° or more from a right angle, additional signing may be needed to clearly mark the through route, particularly for 3-legged intersections.

Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, and NCHRP 279 *Intersection Channelization Design Guide* provide more detailed information on Y-type intersections.

### D. Four-Leg

Four-leg intersections are one of the most common intersection types. There are numerous variations involving channelization, traffic control, skew, and number of through and turning lanes. Refer to Sections 5.9.2 through 5.9.8 of this chapter for guidance and requirements applicable to these intersections.

### E. Multileg

Multileg intersections are those with 5 or more intersection legs and should be avoided whenever practical. Refer to Sections 5.9.2 through 5.9.8 of this chapter for guidance and requirements applicable to these intersections.

#### 5.9.1.3 Nontraditional Intersections

Nontraditional intersections require special consideration and treatment, and they should be developed in consultation with the Regional Transportation Systems Operations Engineer. Additional information, including the layout, applicability, design features, safety performance, and operational performance of the following intersections discussed below are covered in FHWA's *Signalized Intersections: Informational Guide*.

#### A. Jughandle (Indirect Left Turns)

Where operational or safety concerns preclude left turns from the median lane, indirect left turns or jughandles can, if adequate advance signing is provided, provide safe and efficient left-turn access by diverting left turns to separate turning roadways which cross the mainline or intersect the cross street at a different location. Refer to Exhibit 5-25.

## B. Quadrant Roadway Intersection

A quadrant roadway intersection includes an extra roadway between two legs of the intersection. The roadway is bidirectional, forces left turning traffic to travel a greater distance, and creates two T intersections that can operate with a three phased signal. The design removes all left turns from the major intersection, which can be signalized with a 2 phase signal, greatly increasing the green time for the through movements. A key element of this design is to locate the quadrant roadway a sufficient distance back from the major intersection to eliminate the potential of queue spillback.

## C. Others

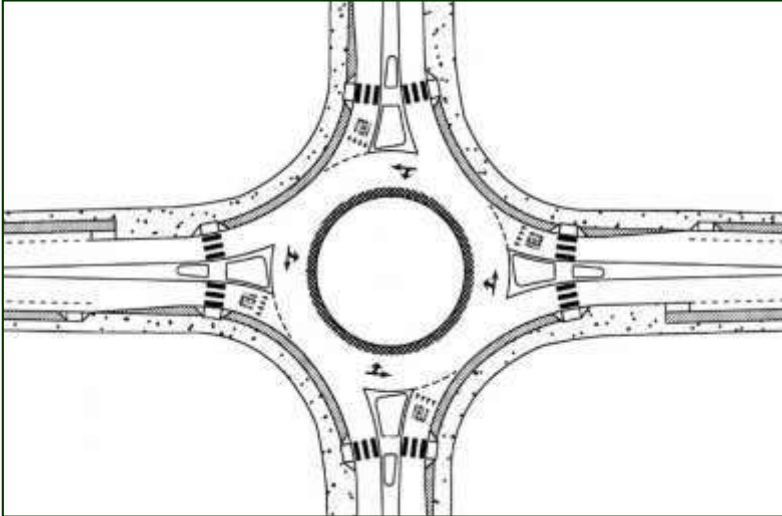
Several other nontraditional intersection types have been developed. They include:

- Median U-Turn Crossover
- Continuous Flow Intersection
- Super-Street Median Crossover



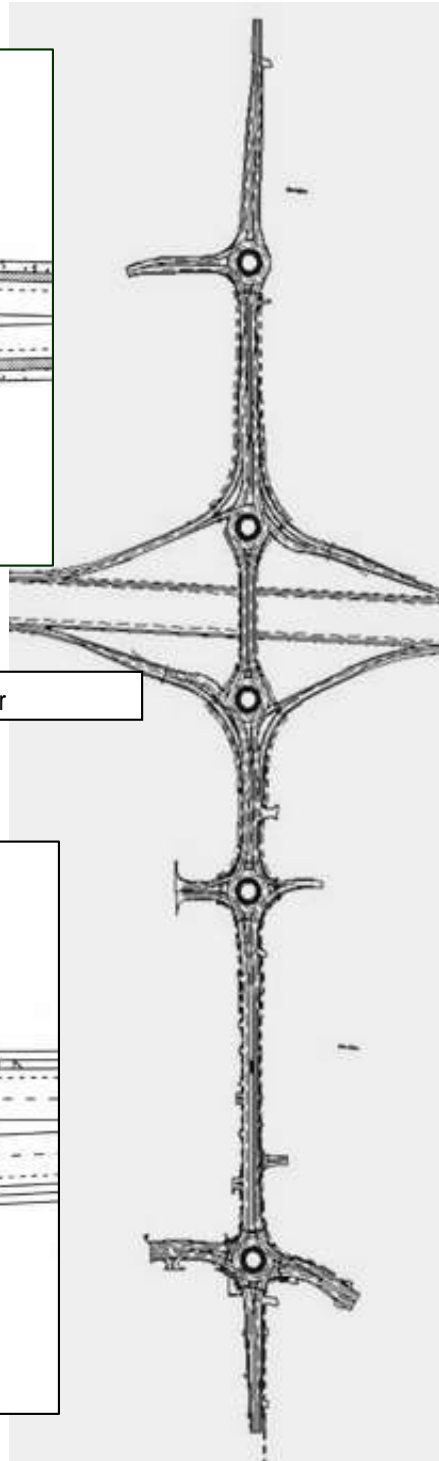
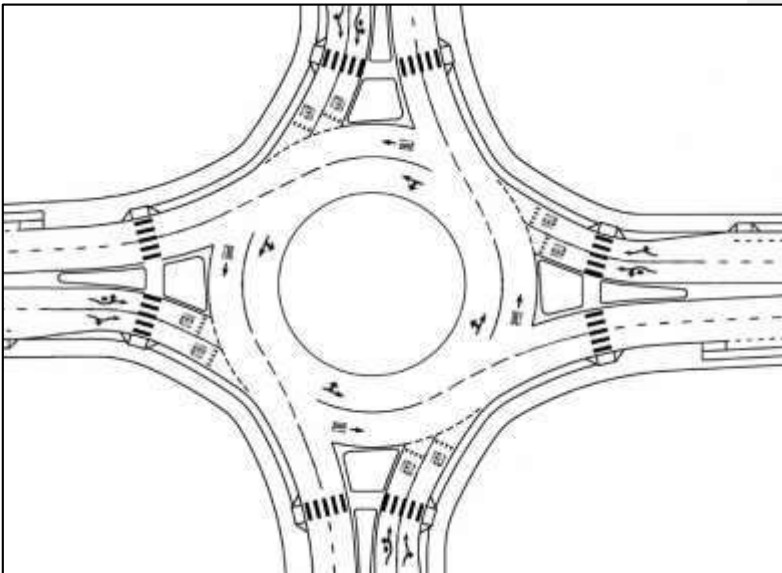
**Exhibit 5-23 Roundabout Intersections**

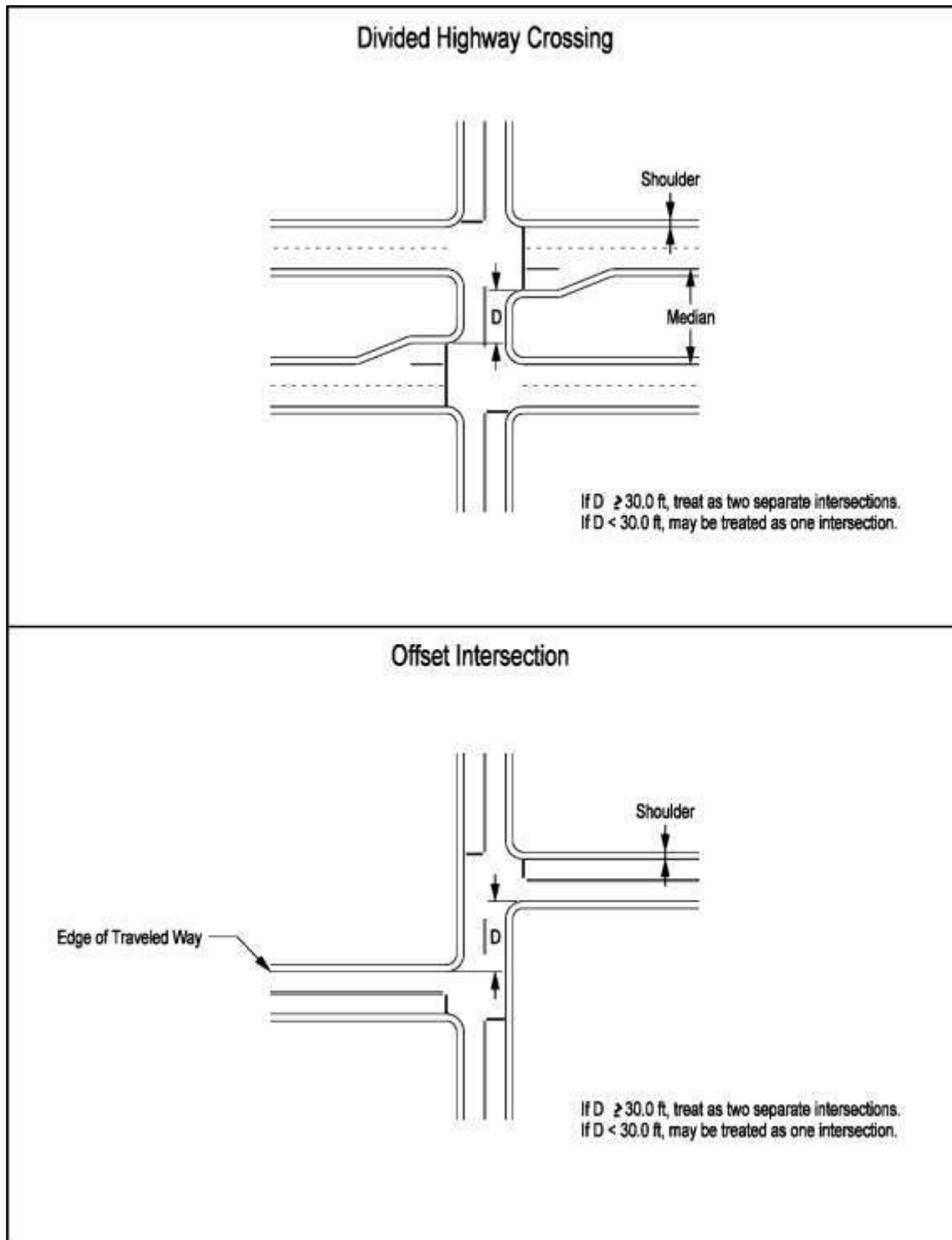
A: Example Single Lane Roundabout

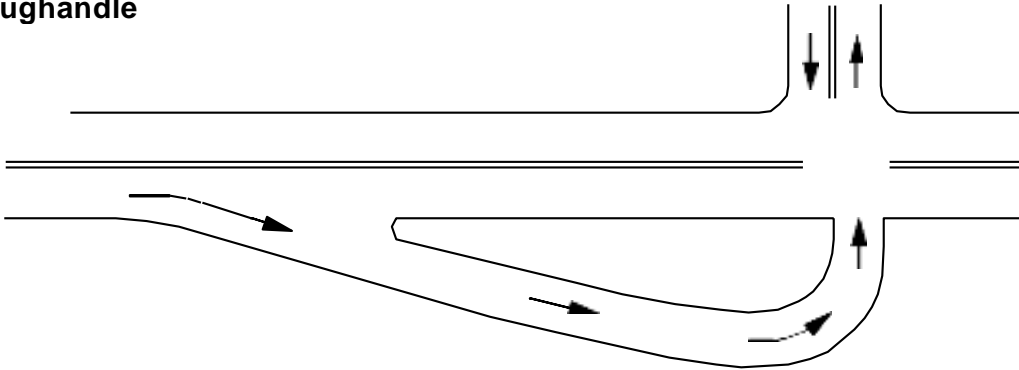
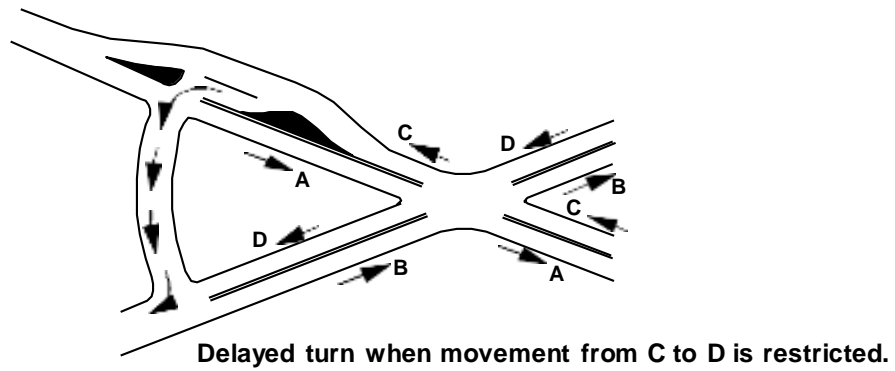
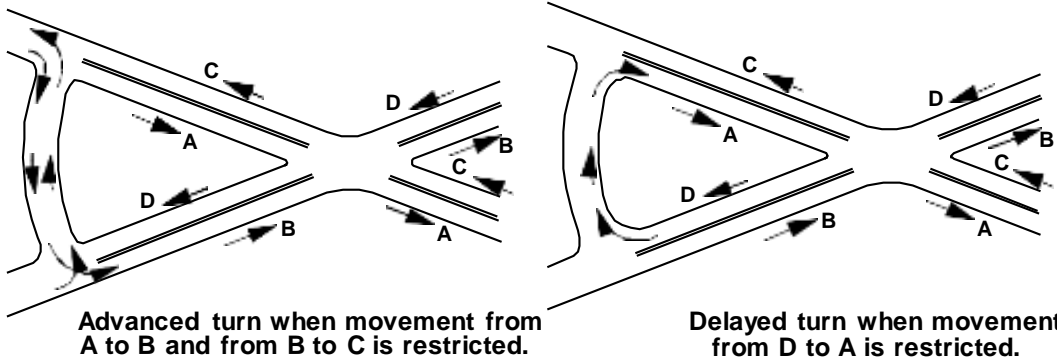


C: Example of Roundabout Corridor

B: Example Multi-lane Roundabout



**Exhibit 5-24 Divided Highway Crossings and Offset Intersections**

**Exhibit 5-25 Jughandles and Indirect Left Turns****T-Intersection with Jughandle****Indirect Left Turns**

## 5.9.2 Intersection Capacity and Level-of-Service Analysis

### 5.9.2.1 Motorized Traffic














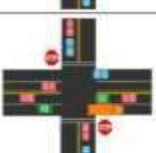
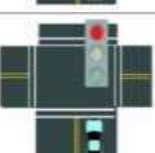

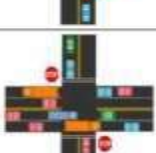
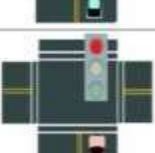
The *Highway Capacity Manual* (HCM) quantifies the quality of traffic flow in terms of levels of service (LOS). As indicated in Section 5.2.3.4, there are six levels of service with LOS A representing very low levels of delays and F representing high levels of delays associated with congestion. Level of service should be a consideration for every project except preventive maintenance projects. The intersection design elements and traffic control techniques selected should meet the level of service objective.

Levels of service and capacity for signalized intersections are calculated for each lane group (a lane group may be one or more movements), each intersection approach, and the intersection as a whole. The intersection level of service is merely a weighted average of the individual approaches and may not be considered a valid measure of the quality or acceptability of an intersection design since it can conceal poor operating conditions on individual approaches. It is a common error to consider an average intersection LOS C as acceptable while one or more lane groups are at LOS F. Correct intersection design practice strives to provide design-year LOS D or better on each lane group in urban areas and LOS C in rural areas.

In some cases, it may be necessary to accept LOS E or F on individual lane groups due to unreasonable costs or impacts associated with improving the level of service. In such cases, acceptance of LOS E or F should be agreed to in the scoping/design process and explained in the Design Approval Document. Note that seconds of delay should be used in design approval documents as it may be easier for the public and decision makers to understand.

Level of service for signalized and unsignalized intersections is based on control delay, as shown in Exhibit 5-26. Control delay is defined as the average vehicle delay in seconds caused by the traffic control device compared to the uncontrolled condition. This includes the delay due to deceleration from the free-flow speed to the back of queue (if any), queue move-up (as needed), stopping/yielding, and accelerating to the free-flow speed.

Exhibit 5-26 Control Delay and Level of Service (LOS)

Level of Service	Flow Conditions	Technical Descriptions	Flow Conditions	Delay per Vehicle (seconds)	Technical Descriptions	Delay per Vehicle (seconds)
<b>A</b>		Highest quality of service. Free traffic flow with few restrictions on maneuverability or speed. <b>No delays</b>		$\leq 10$	<b>Very short delays</b>	 $\leq 10$
<b>B</b>		Stable traffic flow. Speed becoming slightly restricted. Low restriction on maneuverability. <b>No delays</b>		11-15	<b>Short delays</b>	 11-20
<b>C</b>		Stable traffic flow, but less freedom to select speed, change lanes or pass. <b>Minimal delays</b>		16-25	<b>Minimal delays</b>	 21-35
<b>D</b>		Traffic flow becoming unstable. Speeds subject to sudden change. Passing is difficult. <b>Minimal delays</b>		26-35	<b>Minimal delays</b>	 36-55
<b>E</b>		Unstable traffic flow. Speeds change quickly and maneuverability is low. <b>Significant delays</b>		36-50	<b>Significant delays</b>	 56-80
<b>F</b>		Heavily congested traffic. Demand exceeds capacity and speeds vary greatly. <b>Considerable delays</b>		$> 50$	<b>Considerable delays</b>	 $> 80$

(Ref. Exhibit 419-1 and 18-4 of the *Highway Capacity Manual*, 2010.)

Levels of service at unsignalized intersections are only calculated for minor movements since the through movement on the major street is not affected by intersection traffic control. The level of service for signalized intersections and unsignalized intersections can be compared. When a traffic signal is installed, the introduction of delay to the main street usually increases overall intersection delay.

Refer to Section 5.2 of this chapter for a discussion of the traffic analysis software to be used for signalized, roundabout, and other unsignalized intersection capacity analysis. Refer to the roundabout pages on the Department's Internet and IntraDOT sites for requirements and guidance when performing roundabout capacity analysis. Refer to the procedures in the HCM for requirements and guidance when performing capacity analysis for other types of intersections (i.e., the HCM procedure is not to be used for roundabouts). Capacity analysis must be reviewed by someone with expertise in capacity analysis and signal operations to ensure proper modeling of the intersection configuration and the signal operation.

Intersection turning counts for AM, PM, and other peak periods, peak-hour factors (PHF), and the percentage of heavy vehicles are foremost among the data required for the capacity analysis. The PHF, as defined in the HCM, is critical to the analysis, and site -specific data, rather than default values, should be used. Data on the number of pedestrians, location and frequency of bus stops and parking are required for signalized intersection analysis. Section 5.2 of this chapter elaborates on required traffic volume data.

Refer to [Chapter 11](#) of this manual for the detailed design of traffic control devices (i.e., signs, signals, pavement markings, etc.).

#### 5.9.2.2 Non-automobile Level of Service

For high density main streets and central business/walking districts with very high peak pedestrian traffic and/or bicycle volumes, it may be necessary to determine non-automobile delay and LOS. The *Highway Capacity Manual* provides guidelines to determine delay and LOS for both pedestrians and cyclists.

At intersections with high volumes of automobiles and pedestrians and long cycle lengths, it may be useful to calculate pedestrian delay to determine if a pedestrian overpass or a better balance of pedestrian and motor vehicle delay is needed to improve safety. When pedestrians (p) experience more than 30 seconds (s) of delay, they become impatient, and may take greater risks and cross at inappropriate times. At intersections with high conflicting vehicle volumes, pedestrians have little choice but to wait for the walk signal, and observed noncompliance is less (refer to the *Highway Capacity Manual* for more guidance).

#### 5.9.2.3 Balancing Level of Service

Where it is not feasible to simultaneously improve LOS for all traffic modes through design and operational modifications, tradeoffs are necessary. The *Highway Capacity Manual* should be referenced to establish an optimum LOS for each mode that appropriately balances the competing needs of motorists and pedestrians.

### 5.9.3 Intersection Geometrics

When establishing the intersection geometry, designers should recognize the following driver expectations so as to make the navigation and decision making process simpler for the driver:

- The number of through-lanes approaching and leaving an intersection will be the same
- The most important route will be the most direct.
- Left turns from an arterial street will be made from the left-hand-lane, while right turns are made from the right lane and
- What appears to be a through-lane will not be dropped at an exclusive turning lane.

The following discussion applies to traditional intersection designs. Refer to [Chapter 18](#) of this manual for bicycle and pedestrian considerations and refer to the roundabout pages on the Department's Internet and IntraDOT sites for requirements and guidance on roundabouts.

#### 5.9.3.1 Intersection Approaches

Avoid using short radius curves or unnatural travel paths near the intersection (i.e., a hooked intersection), only for the sake of reducing intersection skew. Turning vehicles often follow smoother, more natural travel paths rather than conforming to abrupt alignment changes. Abrupt approach alignment can lead to encroachments into opposing lanes, unwanted detector actuation, prematurely worn pavement markings, crashes, and poor visibility. If poor approach visibility cannot be avoided, provide SIGNAL AHEAD (W2-17) or STOP AHEAD (W2-15) signs on the appropriate approach(es).

The intersection approach curves, where traffic is not always required to stop, must be consistent with the design speed established for each approaching roadway in accordance with the requirements of [Chapter 2](#) of this manual.

Certain intersection types with sharp radius curves or stop conditions require motorists to reduce their speed below the anticipated operating speed. Strict application of the design speed (measured along the open highway) for the intersection approaches may not be needed or appropriate. Therefore, intersection approach curves may be designed with a design speed of 15 mph (20 km/h) below the design speed of the approach highway where all of the following are met:

- The design speed of the approach highway was established in accordance with Section 5.2.5 of this manual.
- The curve will be within 1,000 ft (300 m) of an intersection.
- The curve is on a leg of a roundabout or the minor leg of a T intersection (that is stop controlled or yield/signal controlled with an acute angle of 60 degrees or more).
- The intersection does not violate driver expectancy and adequate sight distance and/or advance warning devices will be used.
- The curvature will not obscure the back of queue during the design hour (i.e., the horizontal sight distance and stopping sight distance are adequate).
- The curve is not using a ramp design speed, which is already less than the highway main line speed.



**Note:** Studies have shown that limiting the change in the design speed to 15 mph (20 km/h) can reduce the crash rate. Additionally, a study specific to roundabouts showed that successive reverse curves prior to a roundabout can also reduce crash rates.

Speed reductions of more than 15 mph (20 km/h) along the approach to an intersection require a nonstandard feature justification since a vehicle's ability to decelerate is diminished when negotiating sharp radius curves. Two and four way stop controlled intersections may use a reduced speed only if justified as a nonstandard feature since these intersection types often lend themselves to future signalization and much higher operating speeds.

#### 5.9.3.2 Highway Alignment Through an Intersection

Abrupt alignment changes within the intersection can lead to encroachments into opposing lanes, unwanted detector actuation, prematurely worn pavement markings, crashes, and poor visibility. The vertical alignment should not place low points within the intersection.

Horizontal alignment shifts are permitted, but not desirable, for traffic entering the intersection from an approach with design speeds of 35 mph (60 km/h) or less. When approach traffic may enter the intersection at speeds over 35 mph (60 km/h), a smooth alignment using tangents or horizontal curves, based on the design speed of the approach, is needed. This accommodates off-peak periods when traffic may move through signalized intersections at the approach design speed.

#### 5.9.3.3 Intersection Angle

A right-angle intersection provides the shortest pedestrian crossing distance and minimizes the duration of exposure to conflicting vehicles. A right-angle intersection also provides the optimal sight line for drivers to judge the relative position and speed of other vehicles (including bicycles) in or approaching the intersection and to view pedestrians approaching or in the crosswalks. However, intersection angles skewed no more than 30° from a right angle typically do not significantly increase crossing distances or decrease visibility and can be a safe, adequate design.

When intersection angles are skewed more than 30° from a right angle, consideration should be given to realigning one or more approaches especially if operational or safety problems can be attributed to the skew. Methods for realigning the approaches are detailed in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Also, refer to Section 5.9.3.1.

#### 5.9.3.4 Pavement Width

Table 2-9 in [Chapter 2](#) of this manual and Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, guide the selection of pavement widths for turning roadways at channelized intersections. The pavement width is dependent upon the size of the design vehicle, the curvature of the roadway, and the number of lanes. At unchannelized intersections where there is minimal space available, the turning path of the design vehicle governs.



#### 5.9.3.5 Superelevation of At-Grade Intersection Turning Roadways

Sharp curvature and short lengths of intersection turning roadways often preclude the development of full superelevation at a desirable rate. The use of compound curves and/or spirals with gradually changing curvature can assist in developing a desirable superelevation rate. [Chapter 2](#) of this manual lists superelevation rates for intersection curves in relation to design speeds. Superelevation runoff and rollover should also conform to Section 5.7 of this chapter.

#### 5.9.3.6 Intersection Cross Slopes

The cross slope at intersecting roadways affects drainage flow patterns, adjacent sidewalks, pedestrian crossings, travel speeds, and safety. Cross sections and contour plans are often needed, especially at major intersections, to ensure a smooth transition to and from the intersection pavement and to ensure they are constructed to drain properly.

##### A. Traveled-Way Cross Slopes at Intersections

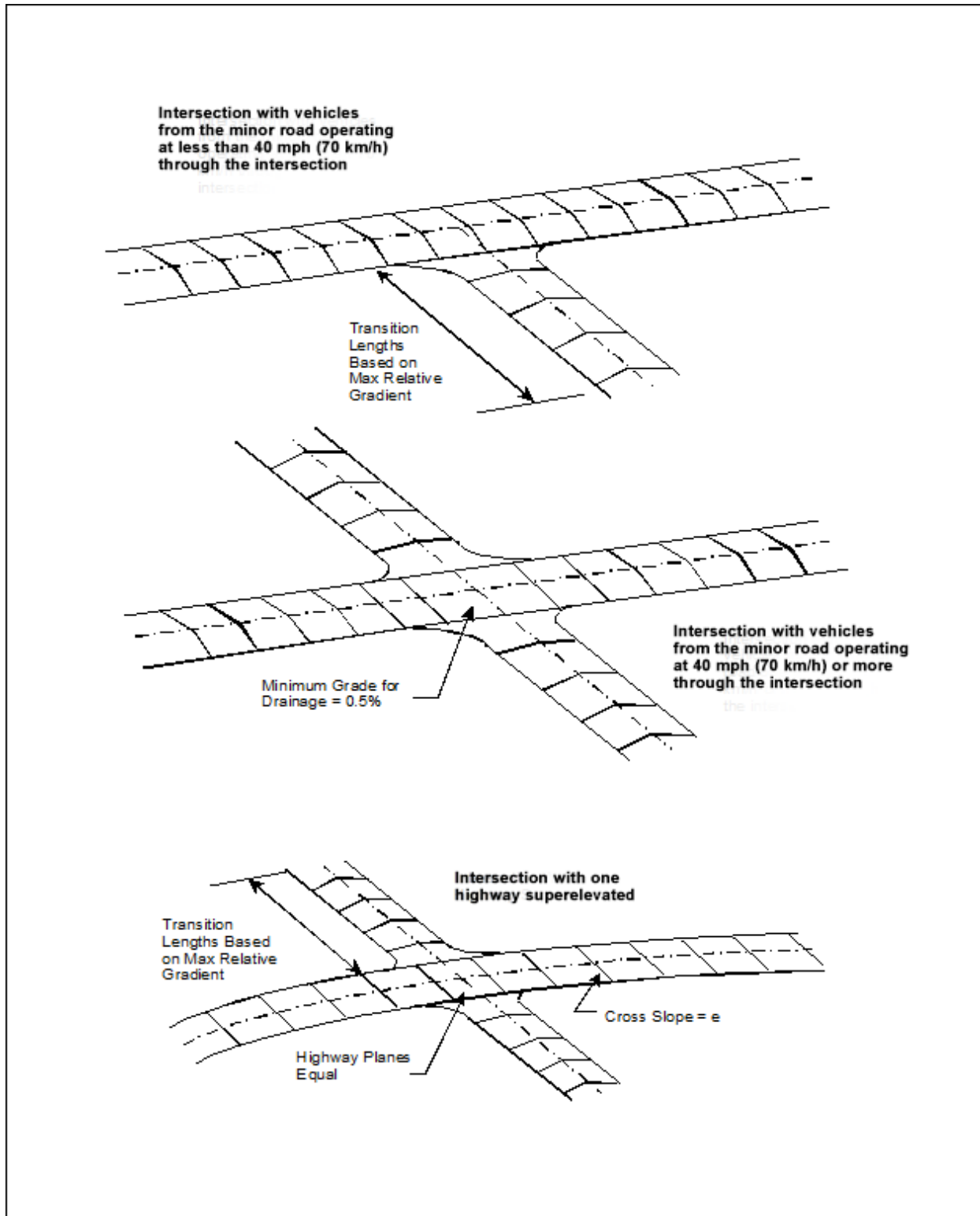
When both facilities are at normal cross slopes, the approach traveled -way cross slope should be treated as follows (Refer to Exhibit 5-27). Note, the cross slope should be designed for design year conditions. For example, the cross slopes of existing stop controlled intersections that will likely become signalized before the design year should be designed as signalized intersections.

- If the off-peak 85<sup>th</sup> percentile vehicle from the minor approach(es) will stop or travel less than 40 mph (70 km/h) through the intersection, normal cross slope should generally be retained on the major highway. The edge of traveled way along the minor approach(es) should be adjusted, using the maximum relative gradient from Section 5.7.3.3 of this chapter, to achieve a cross section that matches the edge of traveled way along the major highway. Vertical curves are to be used to adjust the vertical alignment. A 4% maximum algebraic difference in grade may be used for the minor road crossing at the shoulder breaks and crown-line.
- If the off-peak 85<sup>th</sup> percentile vehicle from the minor approach(es) will travel 40 mph (70 km/h) or more through the intersection, the intersection is to be flattened. A minimum grade of 0.5% should be used to prevent storm water from ponding within the intersection. The edge of traveled way along each approach should be adjusted using the maximum relative gradient from Section 5.7.3.3 of this chapter to achieve an approach cross section that matches the edge of traveled way along the intersecting highway. Vertical curves are to be used to adjust the vertical alignment.

When superelevation is needed through the intersection, an additional 2.0% of superelevation, up to a maximum of 8.0%, may be used to achieve a compatible intersection design. The approach cross slopes for the traveled way should be treated as follows (Refer to Exhibit 5-27):

- If a highway requires superelevation through the intersection, it is to be provided by adjusting vertical alignments and the cross section of the intersecting highway. The edge of traveled way should be adjusted using the maximum relative gradient from Section 5.7.3.3 of this chapter to match the edge of traveled way along the superelevated highway. Vertical curves are to be used to adjust the vertical alignment.
- If two intersecting roadways require superelevation, one of the curves should be relocated. In extreme cases, a broken back curve may be needed.

## Exhibit 5-27 Cross Slopes for Intersecting Highways



## B. Shoulder Cross Slopes at Intersections

Shoulder cross slope rates may be increased or flattened to minimize impacts to adjacent sidewalk and drainage facilities. The maximum rollover rate between the traveled way and shoulder is 8.0%. Desirably, the edge of the shoulder should be adjusted using the maximum relative gradient from Section 5.7.3.3 of this chapter. However, more rapid rates of change may be used to meet site specific constraints.

## C. Slopes of Pedestrian Crossings at Intersections

Grades and cross slopes of pedestrian crossings at intersections are subject to the current Americans with Disabilities Act (ADA) guidelines for accessibility. Refer to [Chapter 18](#) of this manual and the [“Critical Elements for the Design, Layout and Acceptance of Pedestrian Facilities”](#) table for the maximum allowable slopes.

### 5.9.3.7 Intersection Turning Radii

Intersection radii should accommodate the design vehicle turning path. Refer to Section 5.7.1 for a discussion of the appropriate design vehicle. The minimum designs for the inner edge of pavement for turning paths should conform to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Any combination of radii, offsets, and tapers which will approximate the results of the AASHTO designs may be used. The design should consider both the need to keep the intersection area to a minimum and the types of vehicles turning.

If curbs are used, flatter curves provide more room to maneuver. Depending on the intersection angle and design vehicle, asymmetric three centered compound curves will generally reduce the area of the intersection over simple curves with or without tapers. Refer to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, for layouts of three-centered compound curves.

The effect of curb radii on design vehicle turning paths and crosswalk length is shown in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. For larger vehicles turning through more than 90°, three-centered curves or offset-simple curves with tapers or spirals are the preferred design since they fit vehicle paths better and require less pavement area than simple curves do.

Radii design on urban and suburban streets must consider the needs of all users including pedestrians, buses, and trucks. An increase in curb radii may result in an increase in the space needed to accommodate pedestrian facilities for persons with disabilities, an increase in crosswalk distances, and an increase in required right of way or corner setbacks. It may be necessary to provide a raised or curbed island for pedestrian refuge, or to offset the crosswalk to reduce crossing distance.

### 5.9.4 Principles of Channelization

Intersections which are skewed and have enlarged corner radii tend to have enlarged paved areas which can result in uncontrolled movements, long pedestrian crossings and unused pavement. Channelization in the form of properly placed flush or raised islands can control traffic movements by reducing the pavement area available. Examples of channelized skew intersections are shown in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018 and NCHRP 279 *Intersection Channelization Design Guide*.

Properly designed channelization increases operational efficiency and safety by separating or eliminating conflict points and delineating travel paths for turning movements. People-moving capacity can be enhanced by channelizing exclusive paths for high occupancy vehicles and transit. The following principles should be applied to intersection channelization:

- Areas of Conflict - Points of conflict should be separated whenever possible and desirable vehicle paths should be clearly defined.
- Raised/Curbed Refuge Islands - Provide safe refuge for pedestrians and other non-motorized vehicle users.
- Prohibited Movements - Undesirable or wrong way movements should be physically discouraged or prohibited.
- Preference to Major Movements or Designated Vehicles - High priority traffic movements should be facilitated.
- Effective Traffic Control - Desired traffic control schemes should be facilitated and desirable or safe vehicle speeds should be encouraged.
- Turning Roadway Alignment and Terminals - Traffic streams should cross at right angles and merge at flat angles and decelerating, stopped or slow vehicles should be removed from the through traffic stream.
- Size of Islands.
- Curbing of Islands.
- Island Offsets.
- Surface Treatment for Raised Islands.

#### 5.9.4.1 Areas of Conflict

Wide paved intersection areas are generally undesirable. The problems inherent in conflicting movements become magnified due to insufficient guidance and the inability of drivers to anticipate movements of other vehicles within these areas. Desirable vehicle paths should be clearly defined. Channelization reduces areas of conflict by using pavement markings or islands to separate and/or regulate traffic movements into defined travel paths.

Large intersection conflict areas are typical of skewed intersection approach legs. Channelization can reduce conflict areas by reducing the angle at which specific flows intersect. Indirect left-turn roadways and jughandles (refer to Exhibit 5-25) can enhance safety and capacity by separating left-turn conflicts from the rest of the intersection. Refer to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, for more guidance and example configurations.

Where large design vehicles must be accommodated by a wide pavement area, it is common practice to stripe the pavement area for the turning path of a car and allow the larger vehicle to drive over the striping. This practice helps discourage erratic maneuvers by cars or any tendency for cars to form more than one travel lane.

#### 5.9.4.2 Prohibited Movements

Specific movements which are, depending on traffic volume, speed, and other conditions, undesirable from a safety or operational perspective should be prevented or discouraged by channelization. Examples of such movements may include, but are not limited to, left turns out of driveways or side streets, and wrong-way turns. Raised islands or judicious design of curb radii are particularly effective in discouraging prohibited movements.

#### 5.9.4.3 Preference to Major Movements or Designated Vehicles

Directing free flowing alignment to favor major movements should be considered. Major right-turn movements are often given such priority by channelizing them away from the intersection proper and providing separate traffic control as shown in Exhibit 5-28 and discussed in Section 5.9.4.6.

The channelized path should conform to natural paths of the movement, should be introduced gradually to eliminate any surprise or abrupt movements, and should provide adequate turning width and radii for the design vehicle. Exclusive through lanes, turning lanes, and turning roadways (i.e., by-passes) can be channelized, as a component of a comprehensive plan, to give priority to designated vehicles such as buses, high-occupancy vehicles, taxis, carpool vehicles, and bicycles.

Turning roadway elements (e.g., width, radii, and superelevation) are to be designed in accordance with [Chapter 2](#), Section 2.7.5 of this manual and Section 5.9.4.6 of this chapter.

#### 5.9.4.4 Refuge Islands

Raised or curbed traffic islands can enhance pedestrian safety by providing a refuge area. Refuge areas can permit two-stage crossings which can improve traffic signal efficiency by allowing the time allocated for pedestrian movements to be reduced. The width for an island serving as a pedestrian refuge should be a minimum of 6 ft (1.8 m) from the face of curb to face of curb. Curbed islands should be delineated to enhance nighttime visibility. Approach end treatment (e.g., offset, flare, height transition, ramping) should conform to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. All islands in pedestrian paths, whether curbed or uncurbed, must be accessible to persons with disabilities. Refer to [Chapter 18](#) of this manual for design guidance on pedestrian refuge islands.

The decision to use a curbed island should consider the following:

- Anticipated number and frequency of pedestrians using the island, and vehicular volumes
- Pedestrian crossing distance (a refuge should be considered for crossing distances exceeding 60 feet, as discussed in the *AASHTO Guide for the Planning, Design and Operation of Pedestrian Facilities*, 2004).
- Pedestrian exposure to continuous vehicle turning movements during peak/holiday periods
- Potential hazard that curbing may pose to errant vehicles.
- Potential traffic calming benefits to both pedestrian and vehicle traffic.

#### Exhibit 5-27a Curb and Barrier Treatments for Pedestrian Refuge Islands<sup>1</sup>

Design Speed	Curb	Barrier
≤ 40 mph	6" vertical face (non-mountable) curb	Consider low deflection barrier for both curbed and uncurbed refuges for design speeds >35 mph. Barrier should be preferably placed within 1 ft. of the face of curb, but vaulting risk is low.
45 mph -50 mph	4-6" sloping (mountable) curb preferred, 6" vertical face curb allowed	Consider low deflection barrier for both curbed and uncurbed refuges. Due to vaulting concerns, barrier should be placed within 1 ft. of the face of curb.
> 50 mph	4", 1:3 traversable curb or uncurbed (flush-delineated, raised, or depressed island)	See Note 2. Low-deflection barrier system. Impact attenuating end treatments are preferred if barrier is used without curb. Due to vaulting concerns, barrier should be placed within 1 ft. of the face of curb.

**Notes:** 1. In all cases, shoulders or curb offsets from the traveled way should be provided to satisfy the requirements given in [Chapter 2](#) of this manual and Section 5.9.4.8 of this chapter.

2. When design speeds are 50 mph (80 km/h) or greater (high -speed traffic), the frequency and need for pedestrian use must be weighed against the number of motorists and the potential hazard that a barrier system would present to them.

Refer to [Chapter 3](#), Section 3.2, of this manual for a discussion of various curb types and their uses. When barriers and curbing will be used, refer to [Chapter 10](#), Section 10.2.2.4 of this manual for special considerations and requirements.

#### 5.9.4.5 Effective Traffic Control

Proper channelization enhances the effectiveness of actuated traffic signal control at intersections with complex or high volume turning movements by isolating traffic flows which move through the intersection during separate signal phases. Exclusive turning lanes permit efficient use of protected signal phasing.

Exclusive right-turning roadways can, under certain traffic conditions, expedite a heavy right-turn movement by forming a separate yield-controlled intersection with the cross street 30 ft (9.14 m) or more downstream (measured along the travel way edge of the cross street) of the near edge of the intersection of the through lanes. Refer to Exhibit 5-24. The 30 ft (9.14 m) (or greater) separation is measured between the edges of the adjacent roadways as defined by one of the following.

- Pavement marking defining the edges of the travel lanes.
- In lieu of markings, whatever serves as the edges of the travel lanes. The measurement is not dependent upon the type of material existing within the channelizing island area. The island could be flush or raised, paved or unpaved, traversable or nontraversable.
- If the separation between the adjacent roadways is less than 30 ft (9.14 m), the intersection approach and right-turning roadway must be controlled as one intersection, which may negate the benefits of the turning roadway. Consult with the Regional Transportation Systems Operations Engineer to determine the appropriate traffic control.

The Regional Transportation Systems Operations Engineer should also determine if the cross street through movement is light enough to provide sufficient gaps for right-turning traffic during the cross street through movement phase. A heavy, conflicting cross street through movement (volume-to-capacity ratio in excess of 0.85) is not likely to provide additional gaps for right turns other than those provided by the signal. The limited availability of gaps may also result in an unacceptable number of rear end crashes on the turning roadway. Right-turn efficiency and safety can be improved by adding either a through lane or an acceleration lane on the receiving roadway to eliminate the merge or increase the efficiency of merging traffic. To avoid degrading traffic operations and safety, the acceleration lane should be built to standard length per Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

Channelization may require the installation of additional traffic control devices such as yield signs and turn or directional assemblies. At more complex locations, wrong-way signing may be needed but should not be substituted for a design that discourages or prevents wrong-way movements. Provide adequate advance signing for indirect left turns and jug handles. Advance signing is especially important if the indirect left turn is to be executed from the right lane.

Proper channelization can encourage desirable or safe vehicle speeds. Large turning radii and speed change lanes can help reduce the speed differential between turning and through traffic. Small turning radii, stop signs, and oblique entrance angles can reduce vehicle speeds in areas with high pedestrian volumes.

#### 5.9.4.6 Turning Roadway Alignment and Terminals

Channelized right-turning roadways are sometimes called right-turn slip lanes or right-turn bypass lanes. There are two types of channelized right-turning roadways for at-grade intersections: right-turning roadways with corner islands and free-flowing right-turning roadways.



#### A. Right-Turning Roadways with Corner Islands

Right-turning roadways with corner islands are either yield, stop, or signal controlled at the entrance to the intersecting roadway. They do not include acceleration lanes, as shown in the upper left corner of Exhibit 5-28.

The alignment of the edge of traveled way where the turning roadway either diverges from or merges with the through highway should be designed with spirals and/or compound curves. The spirals or compound curves should be long enough to avoid sudden deceleration by drivers while still on the through highway, to provide a natural turning path and to develop superelevation in advance of the maximum curvature. Desirable types of alignments and maximum lengths of spiral for intersection curves and circular arcs for compound intersection curves are shown in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. When turning from a high-speed highway, a deceleration lane is desirable. Deceleration lanes should be designed in accordance with Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

A 90° to 60° angle between the turning roadway and intersecting roadway provides the optimal sight line for drivers entering the highway from the turning roadway to judge the relative position and speed of approaching vehicles. Turning roadways that enter the highway at angles of less than 60° without an acceleration lane require the entering motorist to look over their shoulder to view approaching vehicles and are undesirable.

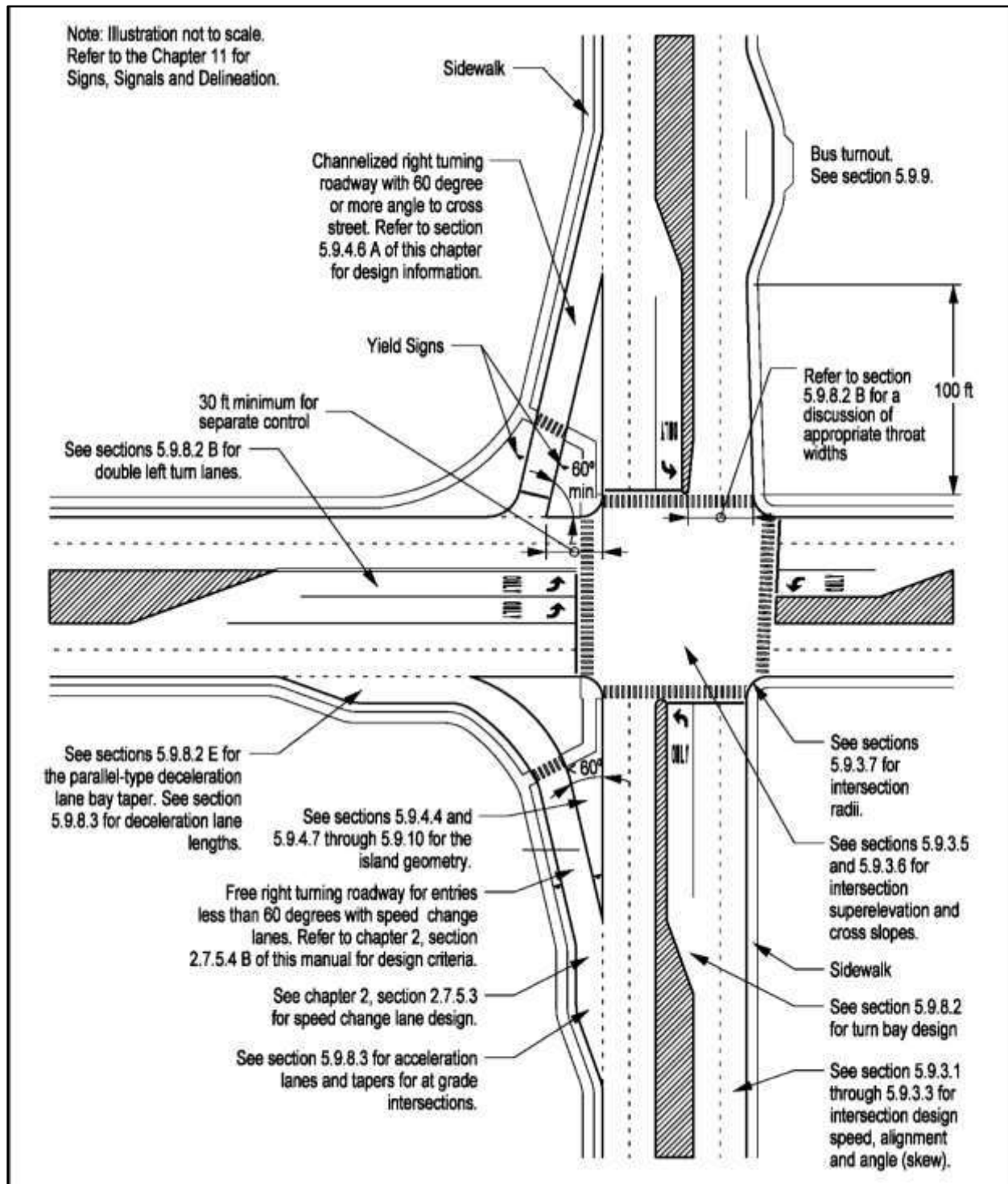
#### B. Free-Flow, Right-Turning Roadways

Right-turning roadways with acceleration lanes are called free-flow, right-turning roadways. They function as ramps for an at-grade intersection.

The alignment of the edge of traveled way where the free-flow, right-turning roadway diverges from the through highway should be designed with spirals and/or compound curves. The spirals or compound curves should be long enough to avoid sudden deceleration by drivers while still on the through highway, to provide a natural turning path and to develop superelevation in advance of the maximum curvature. For high-speed highways, deceleration lanes are desirable.

Free-flow, right-turning roadways should be designed with a near-tangent approach to the highway to encourage use of the acceleration lane, as shown in the lower left corner of Exhibit 5-28. Taper- or parallel-type acceleration lanes are required to allow motorists to use their mirrors to merge into traffic. Acceleration and deceleration lanes should be designed in accordance with Section 5.9.8.3 of this chapter.

**Exhibit 5-28 High-Capacity Signalized Intersection with Double Left-Turn Lanes and Right-Turning Roadways**



#### 5.9.4.7 Size of Islands

Islands should be large enough to effectively channelize traffic flows in advance of the intersection. Small raised islands can lead to maintenance problems and may be difficult for motorists to see and react to. Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, specifies the smallest size curbed islands which normally should be considered. Smaller islands should be flush and color contrasted with the remainder of the pavement. Contrasting surface texture may also be appropriate. The painted area of a wide turning roadway (as described in Section 5.9.4.1) can be included in the legally required minimum 30 ft (9.14 m) separation mentioned in Section 5.9.4.6 and the minimum island area. Islands need to be large enough to accommodate all of the following as appropriate: signs, delineators, pedestrian storage, barriers, landscaping, etc.

Both Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, and NCHRP Report 279 *Intersection Channelization Design Guide* recommend specific dimensions for islands as a function of their use.

#### 5.9.4.7 Curbing of Islands

Refer to Section 5.9.4.4 for a discussion of curbing issues related to refuge islands and islands in general. When it is decided to curb raised islands that are not intended as pedestrian refuges, mountable curbing should be used to allow errant drivers to maintain control of their vehicle. The decision to use a curbed island should consider the potential hazard that curbing may pose to medium and high-speed traffic ( $\geq 45$  mph [70 km/h]). Raised islands bordering high-speed through lanes should be located outside the shoulder area and should use only the 4 in (100 mm) mountable or traversable curbs. Refer to Chapters [3](#) and [10](#) of this manual.

#### 5.9.4.8 Island Offsets

Curbed islands should be offset from the travel lane in accordance with Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Offsets are also recommended for uncurbed islands, but they are not essential. Islands with mountable curbing should be offset from through travel lanes but do not need to be offset from turning roadways unless there is a need to minimize exposure to traffic.

When approach shoulders are used, the shoulders should be continued past the island and the offset between the island and the travel lane should be the shoulder itself except where a deceleration or turning lane is provided. No additional offset from the shoulder edge is necessary, but some may be advantageous at higher operating speeds.

#### 5.9.4.9 Surface Treatment for Raised Islands

Small islands ( $<200$  SF ( $<18$  m<sup>2</sup>)) at intersections should be paved. Easily maintained paving material that cannot be scattered by traffic should be used.

Large islands  $\geq 200$  SF (18 m<sup>2</sup>) may be paved or turfed. Large islands in residential areas may be landscaped provided that the plantings do not interfere with sight distance or grow larger than 4 in (100 mm) in trunk diameter. Islands should not be landscaped without the concurrence of the Regional Transportation Maintenance Group or other maintenance entity if the island is to be maintained by others. Although usually not needed for small islands, large islands should have inlets in the center or along the curbed edges to prevent drainage from adversely impacting adjacent roadways.

### 5.9.5 Intersection Sight Distance

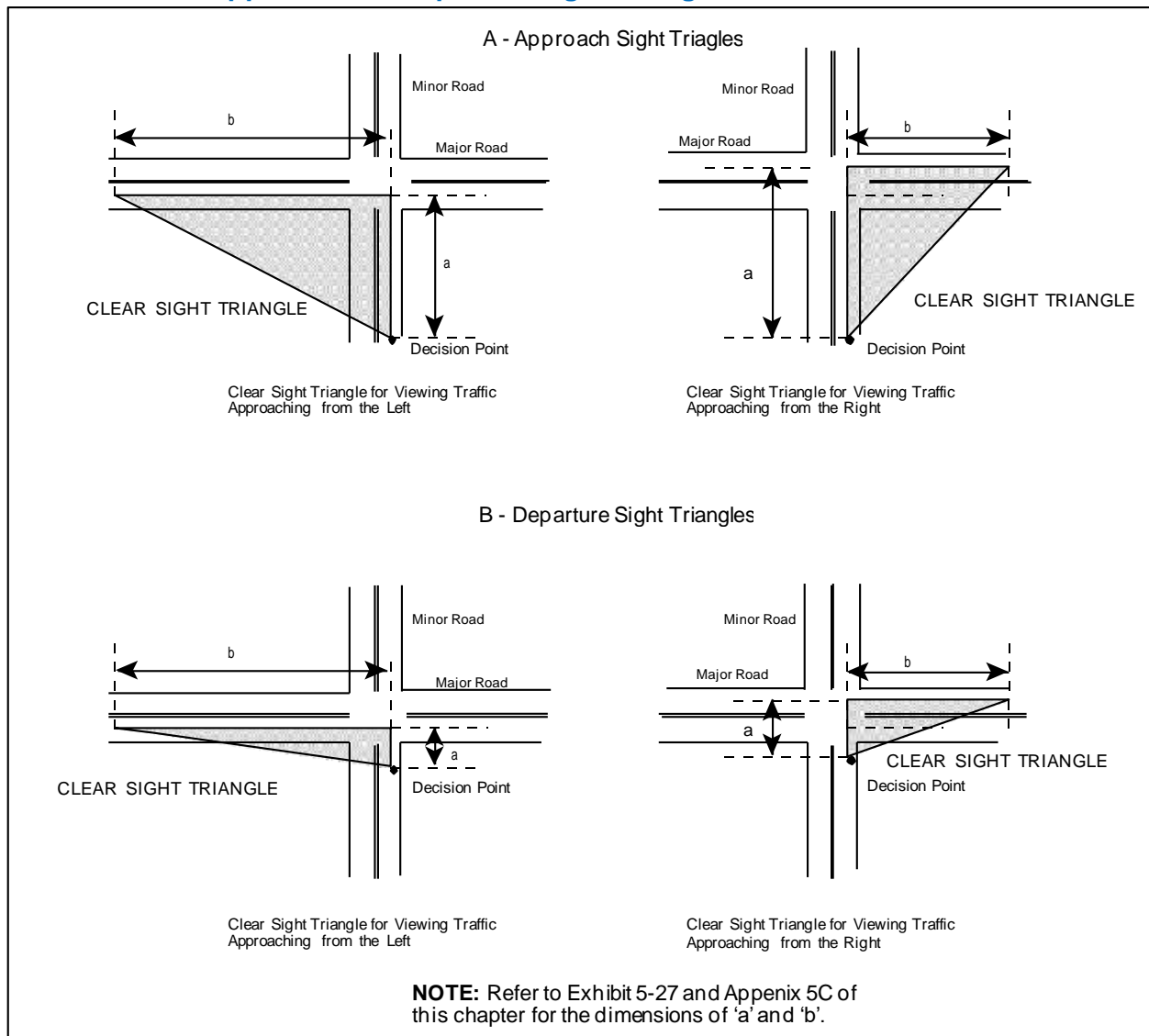
Each intersection has the potential for several different types of vehicular conflict. Providing sight distance at intersections allows drivers to perceive potentially conflicting vehicles. Intersection sight distance should allow drivers sufficient time to stop or adjust their speed, as needed, to avoid a collision in the intersection. The driver of a vehicle approaching an intersection should have an unobstructed view of the entire intersection, including traffic control devices, and sufficient lengths along the intersecting highway to permit the driver to anticipate and avoid potential collisions. Sight distance also allows the drivers of stopped vehicles a sufficient view of the intersecting highway to decide when to enter the intersecting highway or cross it. Sufficient sight distance for motor vehicles also provides sight distances for bicyclists and pedestrians.

**Note:** If the Intersection Sight Distance cannot be met, consideration should be given to adding warning signs or signaling.

#### 5.9.5.1 Sight Triangles

Each quadrant of an intersection should contain a triangular area free of obstructions that might block an approaching driver's view of potentially conflicting vehicles and the presence of pedestrians. These areas are known as clear sight triangles. The intersection sight distance is measured along the "a" and "b" legs of the sight triangle, not the hypotenuse.

The dimensions of the legs of the sight triangles depend on the design speeds of the intersecting roadways and the type of traffic control used at the intersection. Two types of clear sight triangles are considered in intersection design, approach sight triangles and departure sight triangles. The length of the legs of this triangular area, along both intersecting roadways, should be such that the drivers can see any potentially conflicting vehicles in sufficient time to slow or stop before colliding within the intersection. Exhibit 5-29 depicts typical approach and departure sight triangles.

**Exhibit 5-29 Approach and Departure Sight Triangles**

**NOTE:** Refer to Exhibit 5-30 and Appendix 5C of this chapter for the dimensions of 'a' and 'b'.

**Approach Sight Triangle** - The vertex of the sight triangle on a minor-road approach (or an uncontrolled approach) represents the decision point for a minor-road driver. The decision point is the location at which the minor-road driver should begin to brake and stop if another vehicle is present on an intersecting approach. Although desirable at high-volume intersections, approach sight triangles like those shown in Exhibit 5-29-A are not needed for intersection approaches controlled by stop signs or traffic signals.

**Departure Sight Triangle** - A second type of clear sight triangle provides sight distance sufficient for a stopped driver on a minor-road approach to depart from the intersection and enter or cross the major road. Departure sight triangles shown in Exhibit 5-29-B should be provided for stop-controlled and some signalized intersection approaches as discussed in Case D - Intersections with Traffic Signal Control.

The profiles of the intersecting roadways should be designed to provide recommended sight distances for drivers on the intersection approaches. Within a sight triangle, any object that would obstruct the driver's view should be removed or lowered, if practical. Particular attention should be given to the evaluation of clear sight triangles at interchange ramps/crossroad intersections where features such as bridge railings, piers, and abutments are potential sight obstructions. The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, the motorist eye height, and the object height, as shown below:

<u>Vehicle Type</u>	<u>Eye Height</u>	<u>Object Height</u>
Passenger Car	3.5 ft (1080 mm)	3.5 ft (1080 mm)
Single Unit or Combination Truck	7.6 ft (2330 mm)	1.5

#### 5.9.5.2 Intersection Movements

The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection. Exhibit 5-30 provides a quick reference to the procedures for intersection sight distance. Detailed procedures for determining intersection sight distance follow.

#### 5.9.5.3 Intersection Sight Distance Design Guidance

Intersection skew is more of a concern at unsignalized intersections than signalized ones. A traffic signal should not, however, be installed to compensate for intersection skew unless the Regional Transportation Systems Operations Engineer determines that it is warranted (refer to Section 5.9.7). Sight lines between the intersecting highways, even at signalized intersections, are a concern because of right-turn-on-red, flashing signal operation, and power failure.

The intersection sight distance values may be adjusted for intersections skewed at an angle of less than 60 degrees. This adjustment can be made by assuming a greater number of lanes being crossed.

The sight distance of intersections adjacent to bridges can be obstructed or severely limited by bridge railing or approach guide railing. In such cases, sight distance may be improved by relocating the intersection, offsetting the railing by providing a wider shoulder on the bridge and approach or, if practicable, changing to an alternative railing design which optimizes sight distance. Ramp terminal intersections should be designed in the same manner as any other at-grade intersection with the corresponding traffic control.

**Exhibit 5-30 Intersection Sight Distance Quick Reference**

Traffic Control & Maneuver		Traffic Control & Maneuver	Appendix 5C Tables	How to Use Tables in Appendix 5C of this Chapter
Class A - Intersection with no control		approach	A & G	Use table distances for "a" dimension on minor approach legs and "b" dimension on major approaches. Adjust for grade on all approaches using Table G. There are no correction factors for vehicle type.
Class B - Intersection with stop control on the minor road	Case B1 - Left turn from the minor road	departure*	B1	Use table to determine "b" dimension along major road. The "a" dimension is half the receiving travel lane width, plus any median, plus the lane widths being crossed, plus a minimum of 14.5' (4.4 m) for the distance between the driver's eye and edge of traveled way. No adjustment for grade.
	Case B2 - Right turn from the minor road	departure	B23	Use table to determine "b" dimension along major road. The "a" dimension is half the receiving travel lane width plus a minimum of 14.5' (4.4 m) for the distance between the driver's eye and edge of traveled way. No adjustment for grade.
	Case B3 - Crossing maneuver from the minor road	departure*	B23	Use table to determine "b" dimension along major road. The "a" dimension is the distance from the middle of the furthest lane crossed to the outside edge of the traveled way nearest the stopped vehicle plus a minimum of 14.5' (4.4 m) for the distance between the driver's eye and edge of traveled way. No adjustment for grade.
Case C - Intersections with yield control on the minor road	Case C1 - Crossing maneuver from the minor road	approach*	C1 & G	Use table to determine the "a" dimension along the minor road and the "b" dimension along undivided major roads. Use Table G to adjust for grade. For divided roadways, the "b" dimension is based on case B3 for wide medians and B1 for narrow medians.
	Case C2 - Left or right turn from the minor road	approach*	C2	Use table to determine "b" dimension along major road. Use 80' (25 m) for the "a" dimension on the minor road assuming the vehicle enters the intersection at a turning speed of 10 mph (16 km/h) and for left turns, the major road is only 2 lanes wide. (Note that if a stop occurs, the distance "b" based on cases B1, B2, or B3 result in lower values and, therefore, do not need to be checked.) No adjustment for grade.
Case D - Intersections with traffic signal control		8' (2.4 m) from the stop bar on all approaches	Normally none	First vehicle stopped on one approach should be viewable by first vehicle stopped on all others. Permissive left turners should have sufficient sight distance to select gaps. For flashing yellow, use cases B1 and B2 for the minor road approaches. For approaches with right-turn-on-red, use case B2.
Case E - Intersections with all-way stop control		8' (2.4 m) from the stop bar on all approaches	Normally none	First vehicle stopped on one approach should be viewable by first vehicle stopped on all others.
Case F - Left turns from the major road		departure	F	Applies to intersections and left turns into driveways. Check at three-legged intersections and driveways on horizontal curves or crest vertical curves. The "b" dimension is along the major roadway travel lanes being crossed. The "a" dimension is from the eye of the turning motorist to the middle of the furthest travel lane being crossed. Use case B3 when the median width can store the design vehicle length plus 6' (2 m).



### 5.9.6 Access Control on Uncontrolled Access Facilities

When projected volumes approach capacity ( $v/c$  of 0.90 or greater), intersection radii including exclusive turn lanes and jug handles should, if practicable, be protected by acquiring right of way without access. Greater length of access control should be considered if the cost would not increase appreciably.

### 5.9.7 Signalization

The decision to install or modify a traffic signal rests with the Regional Transportation Systems Operations Engineer. Both the [National MUTCD](#) and [NYS Supplement](#) contain warrants for installing traffic signals at previously unsignalized or new intersections. Traffic signals should normally only be installed if one or more of the warrants in Part 4 Highway Traffic Signals, Chapter 4C Traffic Control Signal Needs Studies in the National MUTCD are met and a traffic engineering study indicates that a signal may be justified. A traffic signal is not justified merely because one or more of the warrants are met. The NYS Supplement warrants are based on vehicular and pedestrian volumes, crashes, progressive signal system needs, school crossings, the need for an interruption of continuous traffic on the major road, peak-hour volume, peak-hour delay, four-hour volumes, and systems (to establish traffic flow networks).

Before deciding to build a new signalized intersection or make major improvements to an existing signalized intersection (e.g., reconfigure the intersection, major widening on more than one approach), the alternative of using a roundabout is to be analyzed per Section 5.9.1 of this chapter.

Refer to [Chapter 11](#), Section 11.3 of this manual for requirements and guidance on traffic signals.

### 5.9.8 Intersection Widening

Intersection widening increases intersection capacity and enhances safety by adding auxiliary lanes to serve heavier traffic maneuvers through the intersection. There may, however, be substantial impact to pedestrian traffic as a result of longer pedestrian crossing distances and more complex traffic signal phasing, especially where pedestrian volumes are high. The most common types of intersection widening include addition of exclusive left-turn lanes, exclusive right-turn lanes, and right-turn acceleration lanes. Capacity of the through movement can be increased by adding through lanes upstream of and through the intersection with a downstream taper back to the normal roadway width. Right-turn lanes and acceleration lanes pose special difficulties for bicyclists by requiring them to weave across or merge with higher -speed traffic. [Chapter 17](#) of this manual illustrates a design treatment for right-turn lanes.

#### 5.9.5.3 Additional Through Lanes

Capacity analysis may indicate the need for additional through lanes on the approach to a signalized intersection. The additional through lane(s) must then be carried through the intersection and downstream for sufficient distance to provide a safe merge back into the continuous through lanes as shown in Exhibit 5-31. Since added lanes are generally utilized less than the continuous through lanes, a lane utilization factor should be used. The merge



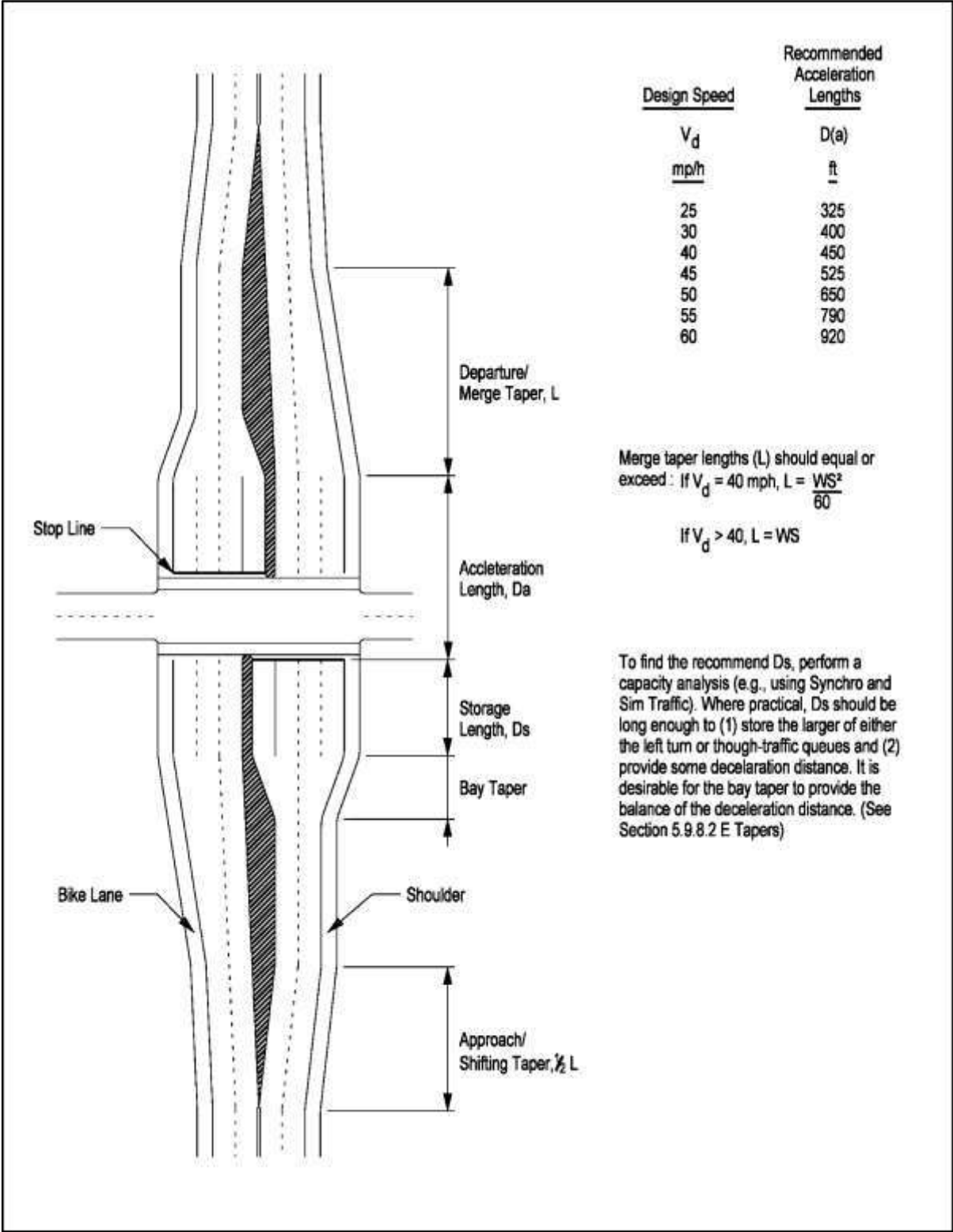
taper on the departure side of the intersection should conform to the length “L” as shown in Table 6H-4 of [MUTCD](#) Section 6H.01. The shift taper on the approach side of the intersection should conform to one half “L” as shown in Table 6H-4. A capacity analysis (e.g., using Synchro and Sim Traffic) should be used to determine the storage length for through and turning vehicles. Both the additional through lane and any exclusive left- or right-turn lane on that approach should be long enough to prevent queues in the through lane from blocking the turn lane entrance and vice versa.

#### 5.9.5.4 Turning Lanes

Exclusive left and right turning lanes increase capacity and enhance safety by removing turning vehicles from the through lanes. This reduces the interference to through traffic associated with vehicles decelerating and queuing in preparation for their turning movement. Exclusive left-turn lanes on multilane highways should always be considered since their absence requires left-turning vehicles to decelerate and/or stop in the high-speed lane. Exclusive left-turn lanes on two-lane highways allow the left-turning vehicle to decelerate and stop without obstructing through traffic.

Turning lane width should be in accordance with [Chapter 2](#) of this manual. Alignment and sight distance criteria should not be compromised for the channelized movement.

To improve operations and sight distance at intersections where the median width is 16 ft (5 m) to 26 ft (8 m), provide a flush divider to the right of the left-turn lane to direct the left-turning vehicle to be within 4' (1.2 m) to 6' (1.8 m) of the opposing travel lane thereby reducing the potential for opposing left-turning vehicles to obstruct each other's view of opposing through traffic. To improve operations and sight distance at intersections where the median width is 6 ft (2 m) to 16 ft (5 m), consider providing a flush divider to the right of the left-turn lane to direct the left-turning vehicle to within 4' (1.2 m) to 6' (1.8 m) of the opposing through lane to reduce the potential for opposing left-turning vehicles to obstruct each other's view of opposing through traffic. Refer to Exhibit 5-35.



**Exhibit 5-31    Intersection Widening for Heavy Through Traffic**

### A. Left-Turn Lanes

The decision to construct left-turn lanes should consider:

- The volume of left-turning traffic and the volume of opposing traffic. In some cases, capacity analysis may clearly indicate a need for left-turn lanes. Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, includes traffic volume criteria to be considered in determining the need for left turn lanes along two-lane highways.
- The crash history. A crash pattern of rear-end crashes involving queued left turners or vehicles turning left in front of opposing traffic is often mitigated by exclusive left-turn lanes. NYSDOT crash reduction factors show an average reduction of around 30% when a left-turn lane is installed and is an appropriate alternative to mitigate a left-turn crash problem.
- The crash potential and the anticipated operating speeds (i.e., the possible severity of a crash).
- Sight distance on the mainline affecting the ability to see a vehicle waiting to turn.
- The construction costs.
- The right of way impacts.

### B. Double Left-Turn Lanes

Double left-turn lanes should be considered at signalized intersections with high left-turn demands or where a reduction in green time allocated to that left-turn movement can significantly benefit the intersection operation. While capacity analysis identifies the need for and impact of double left-turn lanes, left-turn demands over 300 vph and/or storage needs should trigger consideration of them. Fully protected signal phasing shall be provided for double left turns.

Provide adequate throat width on the approach receiving the double left turns to compensate for off-tracking characteristics of turning vehicles and the relative difficulty of side-by-side left turns. Exhibit 5-28 shows a method of expanding the throat width to facilitate the double left turns. A car and the design vehicle should be able to comfortably turn side-by-side. A 36 ft (11 m) wide throat is desirable for double left turns with turning angles greater than 90°. Narrower throats can be provided for more favorable turning angles. A 30 ft (9 m) throat width may be adequate for 90° turns. In constrained situations with favorable turning angles less than 90°, 25 ft (8 m) throat widths may be acceptable. However, throat widths less than 30 ft (9 m) should normally be avoided since they can restrict turning traffic flow and reduce the operational benefit of double left-turn lanes. On the other hand, excessive pavement width, which can mislead drivers and increase pedestrian crossing times, should also be avoided.

If practicable, the intersection should be designed to allow the double left turn to be executed concurrently with the opposing left turn. This allows the flexibility in the signal phasing to serve the double left-turn movement concurrently with either the opposing left turns or the adjacent through movement. If the turning paths of the double left and the opposing left-turn overlap, the left turns cannot be served concurrently.

Dotted lines, in accordance NYSDOT [Standard Sheet 685-01](#) and the [National MUTCD](#) Part 3, Markings, are the appropriate pavement markings used to separate the two-abreast turning lanes and especially opposing turning lanes. The dotted lines should reflect turning paths and have a gap of between 4 ft (1.2 m) and 6 ft (2.0 m).

The design should prevent through traffic from entering and becoming trapped in the double left-turn lanes. The turning lanes should be fully shadowed wherever possible.

### C. Two-Way Left-Turn Lanes (TWLTLs)

Two-way left-turn lanes (TWLTLs) are flush medians that may be used for left turns by traffic from either direction on the street. The TWLTL is appropriate where there is a high demand for mid-block left turns, such as areas with (or expected to experience) moderate or intense strip development. Used appropriately, the TWLTL design has improved the safety and operational characteristics of streets as demonstrated through reduced travel times and crash rates. The TWLTL design also offers added flexibility since, during spot maintenance activities, a travel lane may be barricaded with through traffic temporarily using the median lane.

TWLTLs can reduce delays to through traffic, reduce rear-end crashes, and provide separation between opposing lanes of traffic. However, they do not provide a safe refuge for pedestrians, can create problems with closely spaced access points, and can encourage strip development with closely spaced access points. Consider other alternatives, before using TWLTLs, such as prohibiting midblock left-turns and providing for U-turns.

TWLTLs should generally be limited to streets with no more than two through lanes in each direction. Seven-lane cross sections will likely cause pedestrian crossings to become too long and left turns very difficult in heavy traffic since oncoming vehicles may limit visibility and left turn opportunities. For six lane sections, a raised median design with a pedestrian refuge area should be considered.

Consider installation of TWLTLs where:

- A crash study indicates that a TWLTL will reduce crashes.
- There are unacceptable through traffic delays or capacity reductions because of left turning vehicles.
- There are closely spaced access points or minor street intersections. A general rule is side road plus driveway density of 20 or more entrances per mile (1.6 km).

When one of the above conditions are met, the site may be considered suitable for the use of a TWLTL. Design guidance and requirements include:

- The desirable length of a TWLTL is at least 260 ft (80 m).
- Consider street lighting in accordance with Chapter 12 of this manual.
- Pavement markings, signs, and other traffic control devices must be in accordance with the NYSDOT [645](#) and [685](#) Standard Sheets and the [National MUTCD](#).

- Provide clear channelization when changing from TWLTL to one-way left-turn lanes at intersections.
- Desirable and minimum widths for the TWLTL design are provided in [Chapter 2](#) of this manual.

#### D. Right-Turn Lanes

The decision to install exclusive right-turn lanes should be based on a comparison, using capacity analysis, of intersection operations with and without the turn lanes. At signalized intersections, exclusive right-turn lanes optimize benefits of right-turns-on-red and protected right-turn movements served concurrently (overlapped) with a crossroad protected left-turn phase. Exclusive right-turn lanes may also be used on high-speed roadways to provide deceleration for right-turning vehicles clear of the through lanes. Right turn lanes may be effective at reducing:

- Rear-end collisions.
- Side swipe and head on collisions with opposing vehicles caused by motorists passing the turning vehicle.

Refer to Section 5.9.4.6 for guidance on channelized right-turning roadways.

#### E. Tapers for Turn Lanes

The length of the widened pavement should provide for turning-lane length and bay taper plus, in the case of left-turn lanes, the approach and departure tapers.

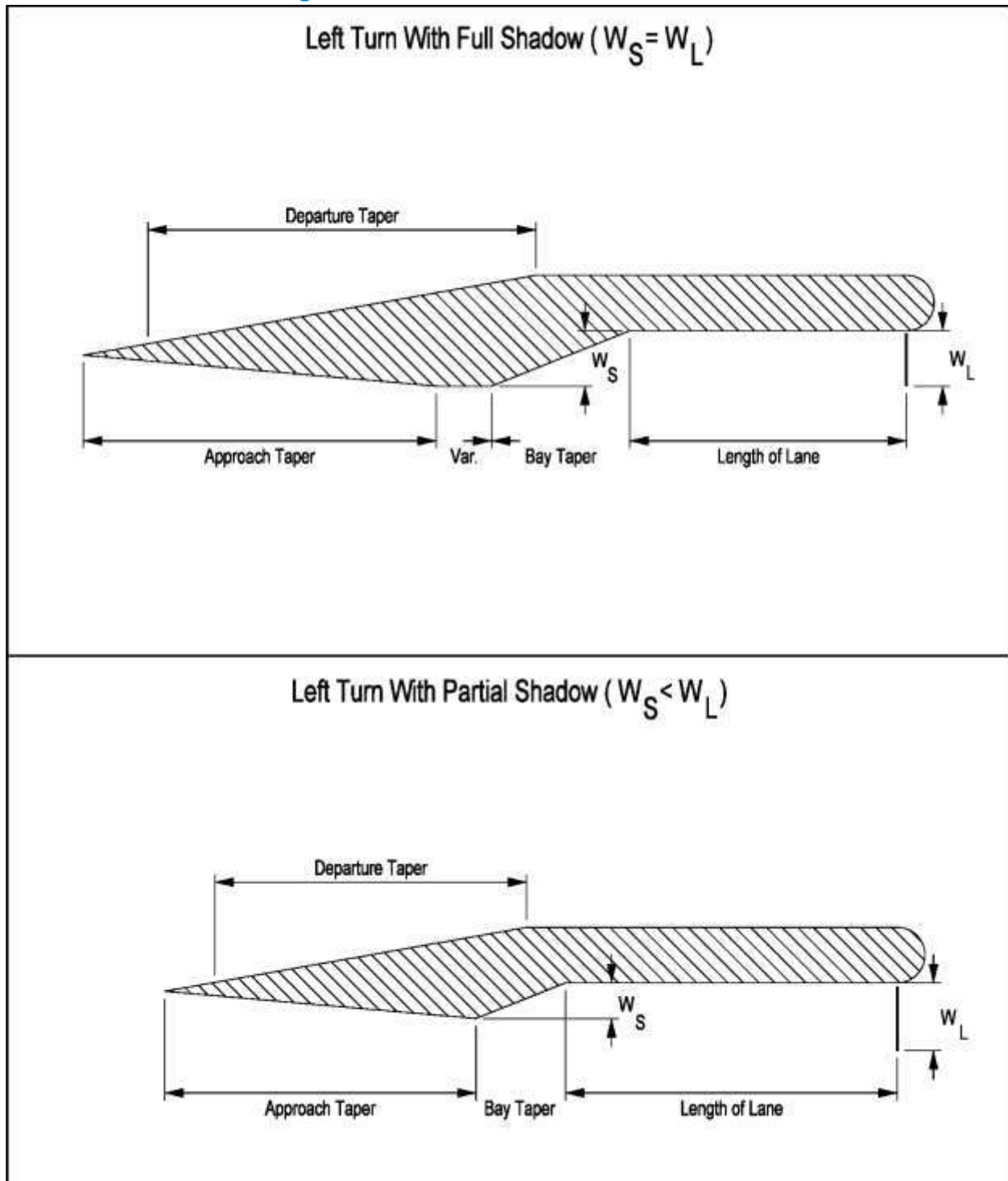
- Approach tapers gradually divert through traffic to the right around the left-turn lane. Approach tapers may be straight-line tapers, may include curves on both ends, or may include a reverse curve. The approach should desirably conform to merge taper requirements in Table 6H-4 of the [National MUTCD](#) Section 6H.01. As a minimum, they should be one half the length “L” determined from Table 6H-4.
- Departure tapers guide through traffic, downstream of the intersection, to the left, back to the normal alignment where the through lane is adjacent to and parallel to the center line. Departure tapers may be straight-line tapers, may include curves on both ends, or may include a reverse curve. The departure taper should desirably conform to merge taper requirements in Table 6H-4 of the [National MUTCD](#), Section 6H.01.
- Bay tapers guide turning traffic from through lanes into the turn lane. Bay tapers should be short enough to enable motorists to identify the widening as a turn lane rather than another through lane. Bay tapers can be straight line tapers, with or without short curves at either end, or they can include a reverse curve as shown in Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. A straight-line bay taper length of 50 ft (15 m) to 100 ft (30 m) is desirable.

Bay taper lengths should not exceed one-half the taper requirements in Table 6H-4 of the [National MUTCD](#) Section 6H.01.

Figure 5-32 shows how the approach taper and the bay taper can be designed to "shadow" or protect the left-turn lane from encroachments by through traffic. In rural and open urban areas, a fully shadowed turn lane permits the complete lateral shift of through traffic upstream of the bay taper. In constrained urban areas, the approach taper and the bay taper can be combined to partially shadow the turn lane by positioning through traffic to continue and complete its shift to the right while the left-turn lane is developing.

While the total turn lane and bay taper length desirably consists of the sum of the required lengths for storage and deceleration, constraints may necessitate assuming some deceleration within the through lanes prior to entering the taper. Deceleration lane length and tapers in rural and suburban areas should conform to Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018. Deceleration lengths in constrained areas (e.g., urban) should conform to Section 5.9.8.2 G. Exhibit 5-32 shows a typical left-turn lane.

Exhibit 5-32 Shadowing Left-Turn Lanes





## F. Queue Storage

To function as designed, exclusive turning lanes must be long enough to prevent queued through traffic from blocking the entrance to the turn lane as well as queued turning traffic from blocking the through lane.

The needs of the individual components of the turn-lane length and their relationship to the total length can vary by time of day. Storage lengths, particularly for left turns, are considerably more complex and depend on the rate of arrivals, rate of departures, and in the case of signalized intersections, cycle length, phasing, and system progression, if any. The desirable design storage length at signalized intersections is twice the length required for the average signal cycle. A minimum of one and one-half the length required by the average cycle should be provided to accommodate surges in traffic which could otherwise cause operational problems which affect subsequent signal cycles.

Double left-turn lanes should be considered for left turn volumes over 300 vph. The storage length for double left turn lanes may be reduced to approximately one half of that needed for a single lane operation unless downstream conditions encourage unbalanced use (e.g., a heavy left or right turn move within 1,000 ft (300 m) of the intersections with the double left-turn.

A capacity analysis (e.g., using Vissim or Synchro and Sim Traffic) should be used to determine both left- and right-turn storage requirements. Since storage requirements are dependent on the traffic signal operation, the Regional Transportation Systems Operations Group should be involved in the design or, as a minimum, the review of the storage lengths. A high percentage of trucks warrants additional storage length.

At unsignalized intersections, desirable storage length should be adequate to store the number of turning vehicles expected to arrive in an average 2 minute period within the peak hour (i.e., 1/30<sup>th</sup> of the peak hour turn movement). All turning lanes should be able to store at least two passenger cars or a car and one truck if there are over 10% trucks in the stream. Assume 74 ft (22.5 m) truck lengths, 18 ft (5.5 m) vehicle lengths, and vehicle spacing of 6 ft (2 m).

- Minimum length for <10% trucks is 42 ft (13 m) = 2 x 18 ft (5.5 m) passenger car + 6 ft (2 m) space.
- Minimum length for >10% trucks is 98 ft (30 m) = 74 ft (22.5 m) truck + 6 ft (2 m) space + 18 ft (5.5 m) passenger car.

## G. Turn Lane Lengths Based on Deceleration Distance for Constrained Areas

In urban and other constrained locations where desirable left-turn lane lengths may result in unacceptable costs or impacts, an alternative design method uses Exhibit 5-33 to determine the deceleration distance for braking at a comfortable rate of 5.8 ft/s<sup>2</sup> (1.8 m/s<sup>2</sup>) from the average running speed. To use Exhibit 5-33, enter the table from the left with the

design speed and go the horizontally to the column for the appropriate speed (usually zero) decelerated to.

For an intersection approach having a design-hour, left-turn volume of 120 vph, 5% trucks, and an 85<sup>th</sup> approach speed of 50 mph (80 km/h), the storage distance would be:

120 vph x 1/30 = 4 vehicles every 2 minutes

4 vehicles require 18' (5.5 m) x 4 veh + 6' (2 m) space x 3 = 90 ft (28 m)

The deceleration distance from Exhibit 5-30 would be 359 ft (109 m)

The total left-turn lane length (exclusive of tapers) would be 449 ft (137 m)

If constraints preclude this length, the turn lane length should be explained as a nonconforming feature.

**Exhibit 5-33 Deceleration Distances (ft) for Passenger Cars Approaching Intersections (Braking at a Comfortable Rate of 5.8ft/s<sup>2</sup> (1.8 m/s<sup>2</sup>))**

Design Speed (Vd)	Running Speed	Speed Reached (Va)														
		MPH	0	5	10	15	20	25	30	35	40	45	50	55	60	65
MPH	MPH	ft/s	0	7	15	22	29	37	44	51	59	66	73	81	88	95
85	67	98	832	828	814	791	758	717	666	605	536	457	369	271	165	49
80	64	94	760	755	741	718	685	644	593	532	463	384	296	199	92	-
75	61	89	690	685	671	648	616	574	523	463	393	314	226	129	22	-
70	58	85	624	619	605	582	550	508	457	397	327	248	160	63	-	-
65	55	81	561	556	542	519	487	445	394	334	264	185	97	-	-	-
60	52	76	501	497	483	460	427	386	335	274	205	126	38	-	-	-
55	48	70	427	423	409	386	353	311	260	200	131	52	-	-	-	-
50	44	65	359	354	340	317	285	243	192	132	62	-	-	-	-	-
45	40	59	297	292	278	255	223	181	130	70	-	-	-	-	-	-
40	36	53	240	236	222	199	166	124	73	13	-	-	-	-	-	-
35	32	47	190	185	171	148	116	74	23	-	-	-	-	-	-	-
30	28	41	145	141	127	104	71	29	-	-	-	-	-	-	-	-
25	24	35	107	102	88	65	33	-	-	-	-	-	-	-	-	-
20	20	29	74	70	56	32	-	-	-	-	-	-	-	-	-	-
15	15	22	42	37	23	-	-	-	-	-	-	-	-	-	-	-

#### 5.9.8.3 Speed Change Lanes for At-Grade Intersections

Speed change lanes minimize the disruption to through traffic from turning vehicles.

##### A. Suitability

The decision of whether or not to provide acceleration lanes should be based on the volume of both through and entering traffic, the intersection geometry, and the 85<sup>th</sup> percentile speed of through traffic. Generally, right-turn acceleration lanes are not

necessary when right-turning volumes are low and the traffic flow being entered has an 85th percentile speed equal to or less than 35 mph (60 km/h). Acceleration lanes should be provided when both through and entering traffic volumes are high and the 85th percentile speed of through traffic is over 50 mph (80 km/h). An acceleration lane may be necessary when right-turn volumes are high regardless of speeds on the intersected highway or the turning roadway intersects the highway at less than 60° as shown in Exhibit 5-28. Acceleration lanes are not usually needed at signalized intersections unless the turning movement is not controlled by the signal.

#### B. Speed Change Lane Geometry

Merging and diverging is most efficient when the angle is small (10° - 15°) and speed differentials are at a minimum. Acceleration or merging lanes should be long enough for merging traffic to attain the average speed of through traffic. Short or nonexistent merging distance can increase the potential for rear-end and other merging crashes. Substandard acceleration lane lengths may be worse than no acceleration lane due to the possibility of violating driver expectancies. AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, should be used to determine the speed change lane lengths. The turning roadway speed should be used to determine the initial speed and the off-peak 85th percentile speed of the highway that the traffic is entering should be used to determine desirable lane lengths. In urban areas or low speed rural areas, the minimum speed change lane lengths should be based on a speed on 15 mph (20 km/h) below the highway design speed.

#### C. Tapers for Speed Change Lanes

For speed change lanes, the deceleration taper should conform the bay taper as described in Section 5.9.8.2 E. Where high speeds are anticipated, the taper should conform to Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018.

#### 5.9.5.3 Safety Widening at Rural Intersections

The potential for rear-end crashes at intersections on high-speed ( $\geq 50$  mph (80 km/h), two-lane rural roads can be reduced by providing a left-turn slot to separate slowing or stopped turning traffic from high-speed through traffic. Safety widening should be considered where:

- The available stopping sight distance is less than the decision sight distance specified in Chapter 3 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, for a stop on a rural road (avoidance maneuver A).
- There is a crash pattern correctable by separating left-turning traffic from through traffic
- Higher traffic volumes increase the time a left-turning vehicle must sit in the travel lane waiting for gaps in traffic; increasing the potential for rear-end collisions.

Since safety widening addresses a speed differential rather than a capacity need, it should, if practicable, provide full deceleration distance from the 85th percentile speed in the total length

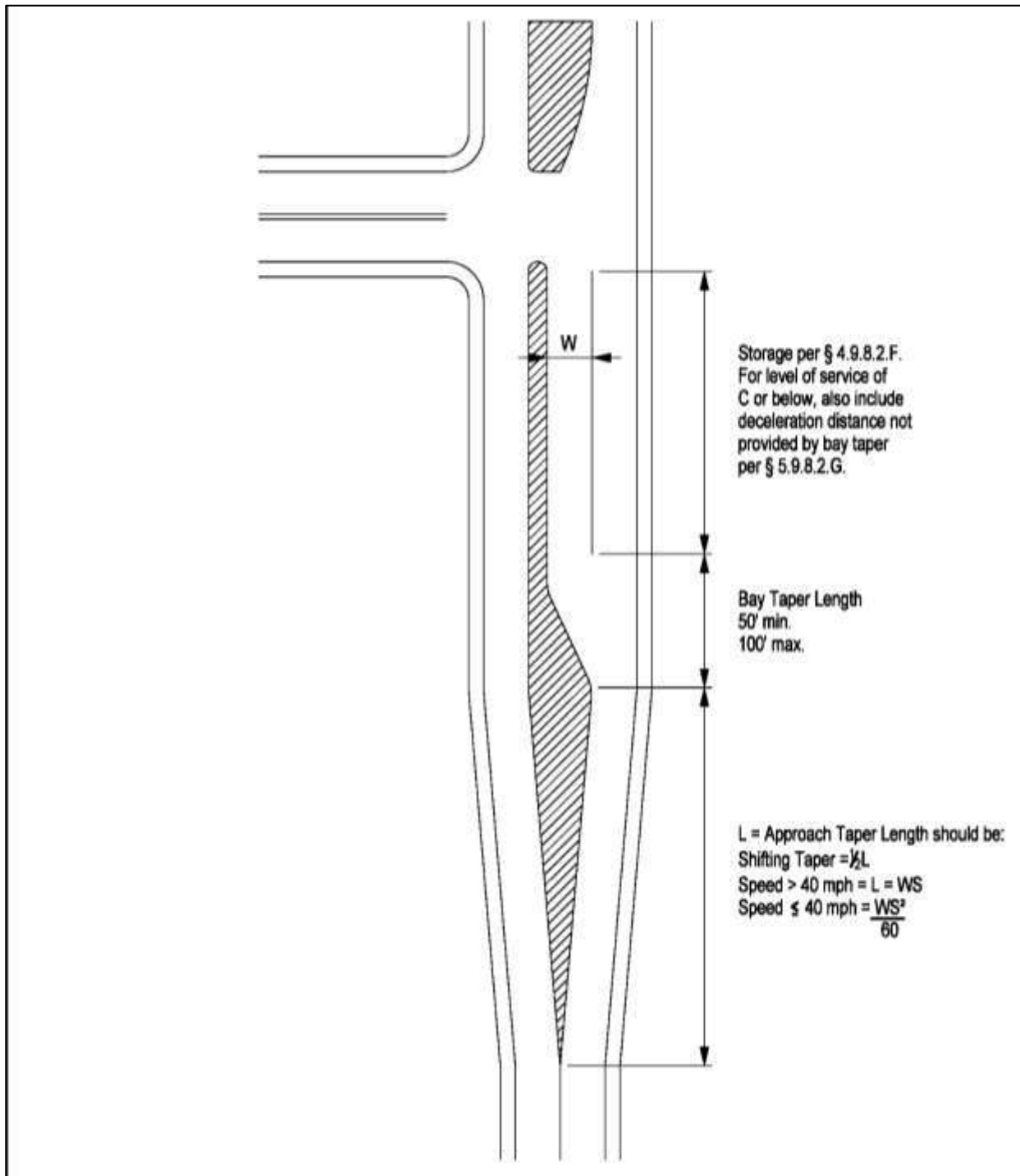
of the bay taper plus the full-width turn lane. Chapter 10 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, or in more constrained conditions, Exhibit 5-33 can be used to determine the deceleration distance required. In addition to the deceleration distance, provide storage for at least one design vehicle. The widening should conform to Exhibit 5-34. If constraints preclude standard taper and deceleration lengths, provide the optimal practical design and document the nonconforming left-turn lane in the Design Approval Document.

The use of a shoulder by-pass lane (i.e., a widened and/or "beefed-up" shoulder that is striped for use by through traffic to go around left turning vehicles) is currently not an acceptable practice and should not be used in lieu of safety widening. This should not be confused with the practice of beefing-up shoulders that remain striped as shoulders (refer to [Chapter 3](#), Section 3.2.5.2).

### 5.9.9 Bus Stops/Turnouts

Bus stops and turnouts should, if practicable, be located at the far side of intersections to facilitate bus and traffic operations. Bus turnouts at intersections with a free right turn should be located 50 ft (15 m) downstream of the end of the right-turn acceleration lane merge taper. Bus turnouts should be provided if the bus stop is on the receiving side of a double left- or right-turn movement and there are only two lanes serving traffic departing the intersection. Refer to Exhibit 5-28 and Section 5.7.21.

Exhibit 5-34 Safety Widening at T-Intersection on Rural Two-Lane Road



### 5.9.10 Divided Highway Median Openings in Urban Areas

Refer to [Chapter 3](#), Section 3.2.8.2 for guidance on choosing between raised or flush medians. The median at an intersection leg should be the same type as on the highway approach. If left-turn volumes require substantial storage for queued left-turning vehicles on an intersection approach which has a continuous two-way, left-turn lane, consider striping the median as a one-direction, left-turn lane far enough upstream of the intersection to provide the storage length required for the left-turning volume.

Raised median openings with left-turn lanes should be provided only at major cross streets and to serve large traffic generators or emergency vehicles. Pedestrian and bicycle travel patterns are to be considered. The designer should, if practicable, avoid opening the median for low-volume (one-way, design-hour volume of 100 vph or less) intersecting streets and left-turn movements from the arterial. U-turn movements should be accommodated at major intersections. The availability of alternate travel paths (e.g., frontage roads) should be considered and may permit elimination of median openings for cross streets with one-way, design-hour volumes over 100 vph. Consider providing roundabouts or indirect, left-turning roadways or jug handles for left turn access if ROW costs and impacts are not excessive. Refer to Exhibit 5-25.

If the median is not wide enough to provide refuge for side-street vehicles crossing one direction of mainline traffic at a time, consider leaving the median unopened if signal warrants are not met. Mainline speeds, traffic volumes, and sight distance are among the factors to be considered in this determination.

Design of median openings should consider the need for traffic to access properties on the other side of the raised median between median openings. Chapter 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, describes design considerations and alternatives for "direct" (from the median left-turn lane at the intersection) and "indirect" U-turns (other than from the median left-turn lane at the intersection). If direct U-turns are to be encouraged, the left-turn lane should be long enough to store both left turn and U-turn traffic. Refer to [Chapter 3](#) of this manual and Chapters 4 and 9 of AASHTO's *A Policy on Geometric Design of Highways and Streets*, 2018, for further design guidance including length of median opening, turning path radii, and median end shape.

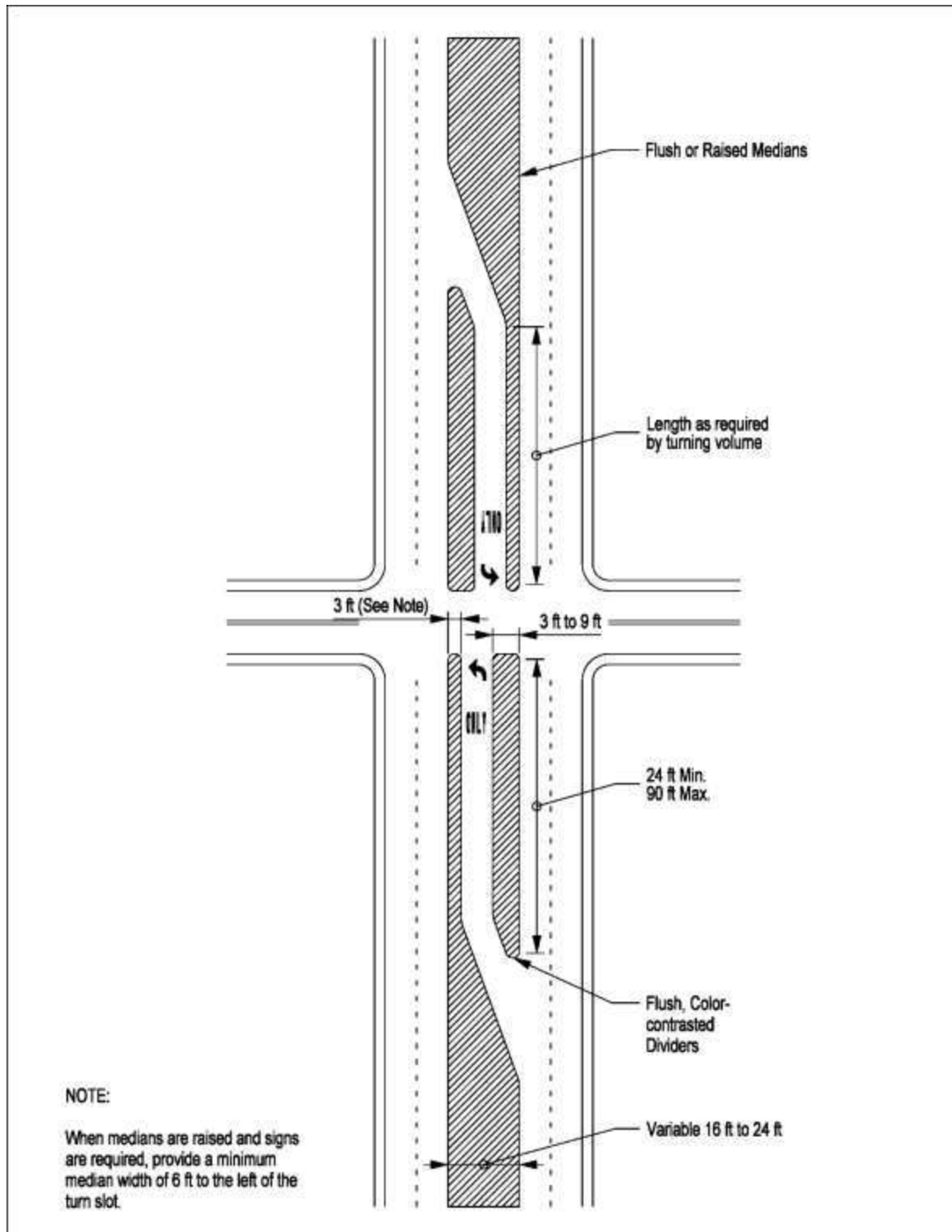
A simple median opening of minimum design which accommodates the design vehicle may be sufficient at minor intersections on a low-speed divided highway with low to moderate traffic volumes. Higher speeds, mainline through volumes, turning demand, and cross street flow require median design adequate to accommodate turning movements with little or no interference between traffic movements.

To improve operations and sight distance at intersections where the median width is 16 ft (5 m) to 24 ft (8 m), provide a flush divider to the right of the left-turn lane to direct the left-turning vehicle as far to the left as possible thereby reducing the potential for opposing left-turning vehicles to obstruct each other's view of opposing through traffic. Refer to Exhibit 5-35.

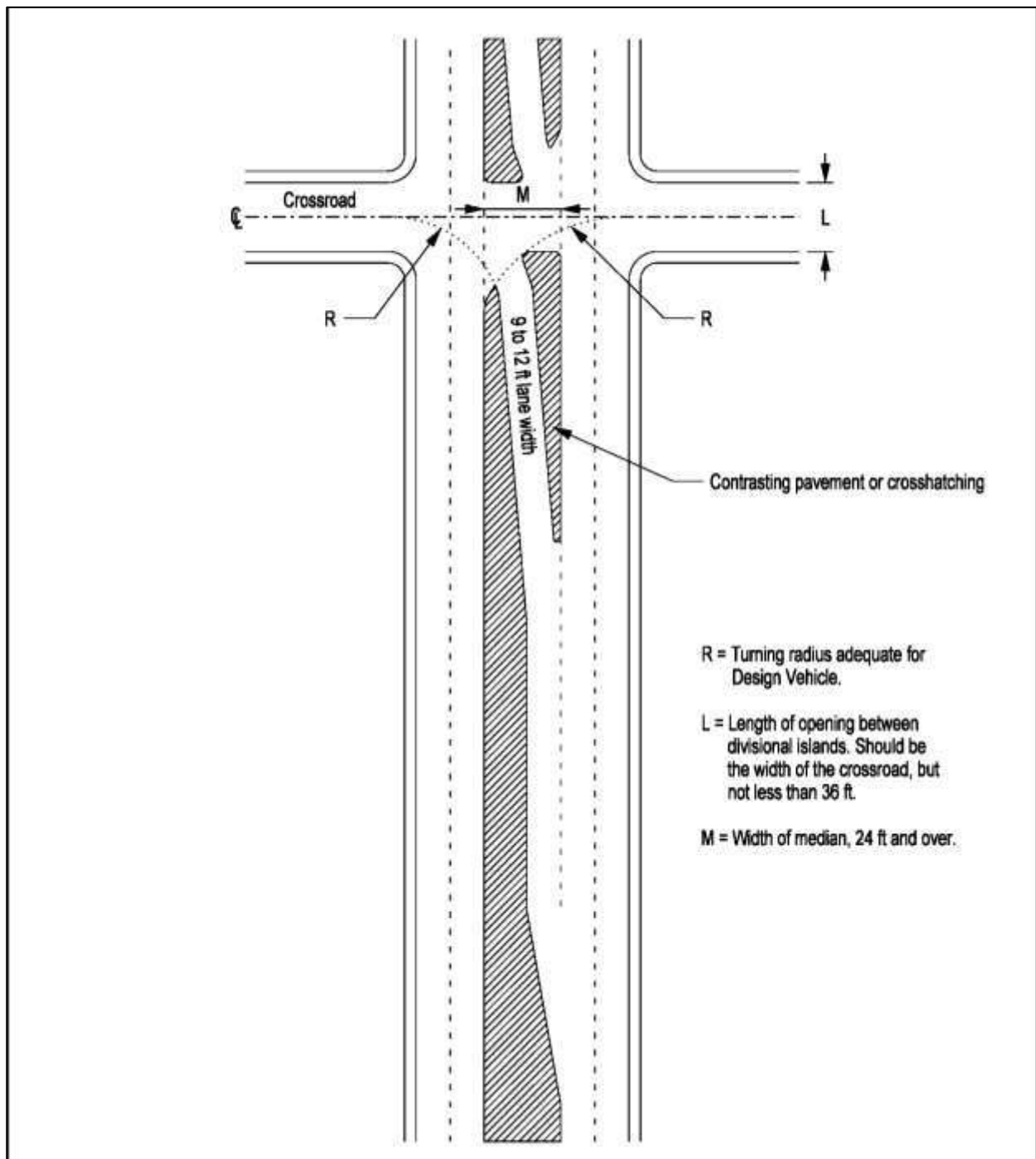
Special attention must be given to all aspects of traffic operations and safety in all medians over 24 ft (8 m) wide. Refer to Exhibit 5-36. Signalized divided intersections where opposing left-turn lanes or the left edges of traveled way are separated by more than 30 ft (9.14 m) should be treated as two intersections with separate traffic signals and/or stop or yield signs. If the median is not wide enough to allow storage of arriving vehicles between the two signals, special signal phasing (double clearances) must be provided to clear turning and cross-street traffic from the median area between the two signals. The design shown in Exhibit 5-36 eliminates the need for two separate signals by locating the opposing left-turn lanes within 30 ft (9.14 m) of each other. This design also provides the flexibility to provide signal phasing which serves both left turns concurrently.

The design of unsignalized divided highway crossings should consider the possibility of eventual signalization or other improvements to the crossing. Safety concerns can result in signalization or reconstruction of high-speed, divided highway crossings long before traffic volumes do.

Exhibit 5-35 Left-Turn Slot with Divider on Right





**Exhibit 5-36 Design of Median Lanes for Medians Over 25 ft (8 m) Wide**

## 5.10 REFERENCES

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